# Blacksnake Creek
# Section 205 Feasibility Study

Geotechnical Report

**CDM Federal Programs Corporation**
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# Section 1

# Introduction

Frequent flooding of Blacksnake Creek within the northwest part of the City of St. Joseph, Missouri, has initiated the Blacksnake Creek Section 205 Feasibility Study, which will ultimately determine a preferred design alternative for flood improvements and mitigation within the basin.

Blacksnake Creek is a small tributary of the Missouri River that runs through St. Joseph, flowing in a predominantly south-southwesterly direction to its junction with the river. The Blacksnake Creek basin includes residential, commercial, and industrial property within the city limits and possesses a total drainage area of 8.2 square miles. Basin flows are conveyed by natural, open channels in the upper five square miles of the watershed, where the creek channel has an average capacity of 5,400 cfs and the floodplain is 400 feet wide with 0.008 feet per foot slope. Typical channel dimensions are 40-feet wide by 10-feet deep.

At Karnes Road, the flows are directed into a combined closed sewer system and conveyed to the Missouri River. On many occasions, the capacity of the combined sewer inlet, which is a 12-foot by 12-foot reinforced concrete box culvert, is exceeded, causing flows to travel south over streets and lawns. At least six major flood events have been recorded at this location, each resulting in major property damage. The City of St. Joseph and The United States Army Corps of Engineers have designated improvements to the drainage basin as a high priority.

This Geotechnical Reports summarizes the design and analyses of geotechnical aspects of the Blacksnake Creek Section 205 Feasibility Study project, including erosion protection and slope stability.

# Section 2

# Geotechnical Analyses

## Review of Geological and Geotechnical Data

Subsurface investigation of the Blacksnake Creek project area was performed by the USACE in January 2004, with geotechnical laboratory testing conducted in July 2004 by Geotechnology, Inc. The data available from this investigation is limited; therefore, many assumptions have been made for the current level of design. It is recommended that the final design contract include a more extensive geotechnical investigation and laboratory testing program.

Site soils are predominantly lean clays. Groundwater was encountered in three of six borings and was measured at levels ranging from 5.3 to 18.2 feet depth which correspond to an approximate elevation (EL) range of 894 to 877 National Geodetic Vertical Datum. For the stability analysis, four separate clay layers were assumed from the geotechnical data: Surface clay extending for a depth of three feet; Medium-Stiff Clay extending to EL 876; Soft Clay to EL 859; and a Stiff Clay layer below EL 859. Table 1 below summarizes the subsurface stratification and shear strength properties assumed for the geotechnical analysis.

**Table 1. Summary of Subsurface Stratification.**

| | | | | Total Stress | | Effective Stress | |
|---|---|---|---|---|---|---|---|
| Stratum | Depth (feet) | Elevation | Unit Weight (pcf) | Phi (deg) | c (psf) | Phi (deg) | c (psf) |
| Surface Clay | 0 - 3 | 897 to 900 | 124 | 0 | 1000 | 32 | 0 |
| Med-Stiff Clay | 3 - 24 | 876 to 897 | 124 | 0 | 870 | 32 | 0 |
| Soft Clay | 24 - 41 | 859 to 876 | 124 | 0 | 410 | 24 | 0 |
| Stiff Clay | 41+ | Below EL 859 | 124 | 0 | 1000 | 32 | 0 |

The USACE Geological and Geotechnical Conditions report, boring logs, and laboratory testing results are included with Appendix A.

## Slope Erosion and Scour Protection

The slopes of the detention basin shall be topsoiled to a minimum depth of four inches and seeded to establish vegetation. Due to the tall height of most of the slopes, temporary erosion control blankets may need to be installed to minimize erosion until the grass is established. Installation of the blankets will be determined by the timing and duration of the slope stabilization. If hydroseeding can be accomplished in a timely manner, blankets would not be necessary. Seed mixture, seeding rates and seeding methods should be coordinated with the City of St. Joseph, with hydroseeding recommended for quicker turf establishment.

Existing soils are typically lean clays, which are considered moderately erodible. Riprap sizing equations used a design velocity of 3.7 feet per second which is the velocity at the bank full condition. The maximum shear stress for the bank full condition on straight reaches of the pilot channel does not exceed the permissible shear stress for a vegetated channel and will therefore not require stone protection. However, the relatively sharp channel bends increase the maximum shear stress in those locations which will require rock riprap on the order of MO-DOT Type II Rock Ditch Liner ($D_{50}$ = 6 inches). A riprap thickness of 2.0 feet is recommended for the pilot channel bends.

On the west side of the proposed detention basin, there are two locations where the basin "juts out" forming a constriction of high flows with power pole islands on the east side of the basin. MO-DOT Type III Rock Ditch Liner Liner (D50 = 12 inches), with a total thickness of 3.5 feet is recommended for armoring these areas.

Riprap is also recommended for downstream of the Savannah Road bridge/culvert. No hydraulic data was available for flowrates and velocities at this location, so a velocity of around 10 feet/second was assumed. Per USACE riprap sizing methodology, this would require a rock with $D_{50}$ around 12 inches (MoDOT Type III Rock Ditch Liner) installed with a thickness of 3.0 feet. The same minimum size riprap and thickness is assumed to protect the existing inlet structure for the proposed detention basin near Karnes Road.

A 6-inch layer of riprap bedding is recommended at all riprap locations. Riprap bedding gradation recommendation is as follows:

- 100% passing 150mm (6 inches)
- 75% to 95% passing 75mm (3 inches)
- 35% to 75% passing 25mm (1 inch)
- 10% to 40% passing 4.75mm (#4)
- 5% to 25% passing 2mm (#10)
- 0% to 10% passing 0.425mm (#40)

Exhibit D-1, which shows the locations and sizes of proposed erosion protection, is included in Appendix D.

## Slope Stability Analysis

Excavation slopes for the Blacksnake Creek detention basin areas were analyzed for global stability using GEO-SLOPE SLOPE/W software. Four separate conditions were analyzed. All reflect a single typical basin excavation slope of 3H:1V from EL 900 to EL 876. Analysis conditions and target factors of safety follow recommendations of USACE Design Manual 1110-2-1902 Slope Stability. Results of the analyses are listed below:

**Total Stress Condition**

Target Factor of Safety (FOS) = 1.30; Actual FOS = 1.30. This condition controls the slope angle. Parameters were assigned based on available unconsolidated-undrained (UU) Q triaxial compression tests, unconfined compression (UC), N-value, and Torvane Shear test data in the 2004 soil boring logs. Cohesive parameters were selected based on a 2/3 rule (2/3 of the data points fall above the assigned value for each stratum). Note that soil borings did indicate data points below the selected parameters, but the 2/3 rule for parameter selection was considered to be a reasonable approach. See Appendix B for additional data on parameter selection.

**Effective Stress Condition**

Target FOS = 1.50, Actual FOS = 1.73. Effective stress parameters were based on Atterberg Limit correlations (EM 1110-2-2504, Table 3-7). Consolidated-undrained (CU) R-BAR triaxial compression test data was available, however, a review of lab data sheets revealed inconsistencies between the table data and stress/strain plots. The results of test data containing inconsistencies were ignored.

**Effective Stress Condition – Rapid Drawdown**

Target FOS = 1.1, Actual FOS = 1.20. Rapid Drawdown of the storage pool from EL. 894 to 876 was assumed for the analysis. A staged drawdown with both effective and total stress input conditions was analyzed.

**Total Stress – Seismic Loading Condition**

Target FOS = 1.10, Actual FOS = 1.21. Horizontal Acceleration of 0.02g was assumed based on USGS Seismic Hazard Map for 10% chance of exceedence in 50 years (peak ground acceleration).

The analyses indicate that 3H:1V is a stable excavation slope based on the available data. If subsequent investigations indicate softer clays than the current data, the slope angle may need to be decreased – at least in localized areas. Since proposed excavation depths will be much less than the assumed 24 feet in most locations, the analysis can be considered conservative. Stability analysis plates are included in Appendix B.

## Northwest Parkway Overflow Section

A 100-year design event was assumed for the erosion protection at the south overflow section on Northwest Parkway. The maximum water surface elevation for this event is El. 898.7, which results in an assumed height of water of 1.7 feet over Northwest Parkway. The corresponding

maximum velocity at the 100-year event is 3.5 feet per second. The roadway overflow section, with a low-point elevation of 897.0, will be overtopped by flows that occur at a frequency between the 25-year and 50-year design events.

USBR riprap sizing calculations for overtopping sections call for large riprap, MO-DOT Type IV Rock Ditch Liner, with a minimum $D_{50}$ of 19" and a total thickness of 3.5 feet on a 8H:1V slope which fits the existing topography. Riprap of the same type and thickness was assumed on the upstream slope of the overflow section also. Pavement for the overflow section should be Portland cement concrete, with a minimum thickness of 9 inches. The concrete paved roadway section should extend for a minimum length of 350 feet, which would provide an extra 50 feet on each side of the 100-year overflow area. It is recommended that a toe-wall be added to the downstream edge of the pavement down to a depth of 4.5 feet to protect against undermining of the pavement structure.

If the City has concerns with the safety and aesthetics of large riprap, alternative erosion protection, such as concrete paving or articulated concrete blocks could be installed. Vegetation could be established along the slope with certain articulated concrete block designs.

## Secondary Overflow Section

At the south end of the proposed detention basin a secondary overflow section is present at the abandoned railroad right-of-way with a top elevation of 898, which would result in the section being overtopped at a higher design frequency event than the Northwest Parkway overflow section although still between the 25-year and 50-year recurrence intervals. MO-DOT Type IV Rock Ditch Liner is recommended to ensure the structural integrity of the overflow section and adjacent roadway bridge embankment. A thickness of 3.5 feet should be installed along the berm slopes and along the natural ground downstream through the confined bridge opening. Approximate extents of the proposed riprap protection are shown on Exhibit D-1 in Appendix D.

## Transmission Towers

For the recommended alternative, the existing electrical transmission poles will be left intact and the excavation will leave berms or "islands" present in the constructed detention basin. The 3H:1V soil slopes are adequate for stability at these locations. Due to the structural importance of the transmission poles, their presence within the detention basin, and their close proximity to the pilot channel, riprap erosion protection should be installed on the berms. The same riprap design assumptions used for sizing erosion protection at the Savannah Road culvert outlet were used in determining the appropriate size and thickness for protecting the power pole islands. A larger riprap size than used on the pilot channel bends, on the order of MO-DOT Type III Rock Ditch Liner ($D_{50}$ = 12 inches), with a total thickness of 3.5 feet should be installed at these locations. Pilot channel slopes within the island areas should also be armored to ensure the channel remains in the established alignment.

## Construction / Water Control Plan

For the purposes of estimating, a flow of approximately 385 cfs, which is the capacity of the proposed pilot channel, was assumed necessary to accommodate during construction. Due to using the existing creek alignment for the new pilot channel, a large temporary diversion channel, capable of accommodating this flow will need to be created during construction. Preliminary sizing indicates a 3-foot-deep channel with a 20-foot-wide bottom and 3H:1V sideslopes will be necessary to handle the construction flow for Blacksnake Creek. Erosion control measures including rock check dams will need to be included in the diversion channel. Temporary diversion of the water from the east tributary will also be necessary, although it is assumed that a 24" or 30" pipe could convey that drainage during construction. From previous study information, Blacksnake Creek base flows -- exclusive of storm events -- are assumed to be around 3 cfs.

A staged construction plan was assumed to facilitate soil stabilization in the basin and to handle the base flows in the diversion. The contractor must consider construction staging, haul road access points within the construction site and haul routes outside of the construction area as major components of his construction plan.

In essence, the majority of the excavation should be done while avoiding disturbance to the existing channel. A basic construction sequencing list was developed and included at the end of this section. Vegetated buffer strips, with maximum construction slopes of 1.5H:1V will be used to separate new excavations from the existing creek.

Erosion control is of the utmost importance during the construction of the Blacksnake Creek improvements. Storm water pollution prevention best management practices (BMP's) will need to be utilized for all aspects of the project. Development and adherence to the storm water pollution prevention plan (SWPPP) is critical. The contractor will need to dedicate personnel to perform storm water management activities.

Groundwater was encountered in boring holes at depths ranging from 5 feet to 18 feet below the surface (El. 900 to 877), which is near the level of proposed excavation. Due to the nature of the clay soils present, it is not anticipated that well point systems will need to be employed during construction. Groundwater, and surface runoff, should be directed to sumps preferably in corner locations that can serve as siltation basins. Sump pumps will be used to pump the excess water to the storm sewer system.

Staging and diversions could be facilitated if a significant portion of the excavation could be accomplished during the late fall and winter months of November through February, when average precipitation amounts are lowest. Blacksnake Creek base flows exclusive of storm events are assumed to be around 3 cfs. Therefore construction that avoids the wettest part of the year – typically April through September – would greatly affect the amount of surface water to be handled.

Final design of the detention basin and pilot channel should account for the benefits of constructing the pilot channel on a new alignment, instead of following the existing creek bed.

Minor realignments of the pilot channel could help greatly with construction staging and directing flows away from the power pole islands.

## Geotechnical Recommendations for Proposed Box Culverts

Since no new box culvert inlet or outlet structures were included in the recommended alternative, geotechnical recommendations for proposed box culverts were not developed.

## Material Quantity Assumptions

- Mo-DOT Type II Rock Ditch Liner – 3,550 CY
- Mo-DOT Type III Rock Ditch Liner – 15,900 CY
- Mo-DOT Type IV Rock Ditch Liner – 10,850 CY

## Preliminary Construction Sequencing

A preliminary construction sequence was developed within the constraints of the provided detention basin and pilot channel alignment. It was included to illustrate assumptions and challenges of construction while accommodating Blacksnake Creek flows.

Construction activities will be greatly influenced by the time of year, proposed alignment, and conditions for erosion control stabilization. For the listed plan, construction was split into two different subareas: Subarea 1, which is the detention basin south of Maxwell Road; and Subarea 2, which lies to the north of Maxwell Road. A construction staging exhibit (D-2) is included in Appendix D. Depending upon available staging areas, the portion of Subarea 1 south of Karnes Road could be split into a separate subarea that could be constructed last.

*STAGE 1 CONSTRUCTION*

1) Install construction signage at road closure locations (Karnes Road, Maxwell Road); warning signs for hauling activities (Karnes Road, St. Joseph Road); and for construction traffic under power lines.
2) Coordinate utility relocations and abandonments in each area of work prior to commencement of construction activities.
3) Establish perimeter erosion control measures, and haul road locations (Karnes Road; Maxwell Road).
4) Establish staging areas: baseball field and parking lot can be utilized until those areas are excavated.
5) Install riprap at the existing combined sewer box culvert inlet as preparation for future diversion discharges.

6) Install stream monitoring gages – upstream of project site for water level; near combined sewer inlet for turbidity and water level.
7) Clear timber & brush from Subarea 1 (below Maxwell Road) and in Creek; Grub only in areas to be graded – do not grub in existing creek. Salvage timber to extent possible.
8) Coordinate dewatering pump locations, conveyances, and discharge locations. Assume ultimate discharge outfall into storm sewer structure.
9) Demolish existing impacted utilities, roadways, parking areas, recreation facilities and related structures in Subarea 1. Leave Maxwell Road grade and culvert intact.
10) Strip topsoil from Subarea 1 detention basin and stockpile on site; stabilize stockpiles if topsoil will be undisturbed for more than 14 days.
11) Excavate Subarea 1 detention basin – keeping a 10-foot wide buffer with 1.5H:1V side slopes between excavations and the existing creek, as well as new pilot channel excavation adjacent to existing creek.
12) Excavate temporary diversion channel to the west of existing Blacksnake Creek; stabilize channel and side-slopes; Preliminary channel sizing calls for a 5-foot-deep channel with a 28-foot-wide bottom and 3H:1V sideslopes. Keep earthen berms between diversion channel and existing creek.
13) Excavate new permanent east tributary channel east of existing Blacksnake Creek; stabilize channel and side-slopes; Keep earthen berms between existing channel and new channel.
14) Stabilize detention basin area and diversion channel area with temporary and permanent erosion control measures. Areas must be stabilized before creek flows are introduced. Tall sloped surfaces may require temporary erosion control blankets.
15) After area is stabilized, divert Blacksnake Creek flows to diversion channel. Install earthen berm at north junction to keep base flows out of existing channel. Do not install separation berm at south junction in order to accommodate east tributary flows.
16) Grub, excavate and grade at new east tributary junction; install pipe crossing across channel to convey and confine east tributary flows across new channel to temporary diversion channel.
17) Install earthen separation berm at existing east tributary to direct flows to new channel and diversion channel.
18) Install earthen separation berm at south junction of existing Blacksnake Creek and diversion channel.
19) Grub, excavate and grade remaining existing/proposed permanent channel.
20) Complete the platforms for power poles; install riprap on upstream slopes and slopes adjacent to the creek channel.
21) Stabilize permanent channel with temporary and permanent erosion control.
22) Remove buffer areas and stabilize.
23) Remove earthen separation berms and divert base flows to the permanent channel location.
24) Fill in diversion channel and stabilize graded area.
25) Remove haul roads to Karnes Road and stabilize graded area.

*STAGE 2 CONSTRUCTION*

26) Clear timber & brush from Subarea 2 and in Creek; Grub only in areas to be graded – do not grub in existing creek.
27) Coordinate dewatering pump locations, conveyances, and discharge locations. Assume ultimate discharge into storm sewer structure.
28) Demolish existing impacted utilities, and related structures in Subarea 2.
29) Strip topsoil from Subarea 2 detention basin and stockpile on site; stabilize stockpiles if topsoil will be undisturbed for more than 14 days.
30) Excavate Subarea 2 detention basin and new channel areas – keeping a 10-foot wide buffer with 1.5H:1V side slopes between excavations and the existing creek.
31) Excavate temporary diversion channel to the west of existing Blacksnake Creek where necessary; stabilize channel and side-slopes; Preliminary channel sizing calls for a 5-foot-deep channel with a 28-foot-wide bottom and 3H:1V sideslopes. Keep earthen separation berms between new channel/diversion channel and existing creek.
32) Stabilize detention basin area, new channel and diversion channel area with temporary and permanent erosion control measures. Areas must be stabilized before creek flows are introduced.
33) After area is stabilized, divert Blacksnake Creek flows to diversion channel in Subarea 2. Install earthen separation berm at north and south junctions to keep base flows out of existing channel.
34) Grub, excavate and grade remaining existing/proposed permanent channel.
35) Complete the platforms for power poles; install riprap on upstream slopes and slopes adjacent to the creek channel.
36) Stabilize permanent channel with temporary and permanent erosion control.
37) Remove buffer areas and stabilize.
38) Remove earthen separation berms and divert base flows to the permanent channel location.
39) Fill in diversion channel and stabilize graded area.

## Recommendations for Additional Geotechnical Investigation

It is recommended that an additional geotechnical investigation and laboratory testing program be conducted to verify the soil stratigraphy and shear strength parameters determined as part of this preliminary feasibility study. Additional investigation into the erodibility of the soils is also recommended as part of the additional investigation.

It is recommended that the geotechnical investigation consist of approximately 15-20 hollow-stem auger borings, each with a depth of 50 feet, or refusal, whichever occurs first. Soil boring locations should be selected based on critical components of the project. For example, transmission tower pads, inlet and outlet locations, overflow embankments, and tall or otherwise critical embankment slopes. Additionally, soil borings should be appropriately spaced around the perimeter of the project to provide adequate information about stratigraphy changes from one end of the project to the other.

It is recommended that soil borings include shelby-tube (for cohesive materials) and split-spoon (for granular materials) sampling at 5-foot intervals for the full depth of the soil borings. A laboratory testing program would then be developed including moisture content testing, Atterberg limit testing, unconfined compression testing, and unconsolidated undrained and consolidated undrained with pore water pressure triaxial compression testing. The tests should be well-distributed throughout the soil boring depth to provide sufficient testing within each encountered stratum.

Since the project involves excavation of material and not placement or embankment construction, there is no anticipated requirement for consolidation testing or standard proctor testing.

# Appendix A

# USACE Geological and Geotechnical Conditions Report, Boring Logs and Laboratory Testing Results

## GEOLOGICAL AND GEOTECHNICAL CONDITIONS

### 1. Regional geology

The northern Missouri area was glaciated during Nebraskan and Kansan time. Most of the ground cover of the Tertiary and Quaternary Periods consists of alluvial clays, sand and gravels, glacial till and gravels, and clays and loess. Alluvial and eolian deposits from the middle to late Pleistocene are extensive in northern Missouri. Southwest of the project site in Wyeth Park Pleistocene deposits have been measured up to 120 feet thick. The measured section is made up of Bignell Silt, Peoria Silt, Farmdale Silt and Sangamon Soil, underlain by Nebraskan silt.

The Pennsylvanian age bedrock of the Douglas Group is exposed along the river bluffs in St. Joseph. The Douglas Group consists mostly of shale, but includes sandstone and limestone layers. Basal limestones of the Shawnee Group outcrop in the hills of St. Joseph above the Douglas group.

Buried river valleys from ancient drainage formed by several periods of previous glaciation have been mapped by inference in the area by Dreeszen and Burchett. There is buried drainage from a major tributary under the southern part of St. Joseph and one just north near Amazonia that join together east of St. Joseph and continues to the present day Grand River valley. These ancient tributaries are capable of producing large quantities of potable water.

The silts and silts with clay encountered in the overburden were easily compressed either during the push or during the extrusion process. The compressed samples did not have a lot of moisture in the matrix. The Torvane readings taken along the sides of the sample were higher in the compressed samples than the uncompressed samples. The silts and clayey silts that did not compress had a higher water content in the matrix and lower Torvane readings along the sides of the sample. Since water is not compressible, it appears that there are a lot of void spaces in the silts allowing compression. Unconsolidated soils, such as slough from a higher slope, would have more void spaces due to lack of compaction at the time of deposition. Settlement could be inferred from the compressibility of the samples.

### 2. Topography

The uplands in the area generally are strongly sloping to steep and highly dissected. They slope mainly to the southeast, with the gradient of the slopes decreasing from west to east. The elevations of the uplands are about 950 feet above sea level, and 765 feet of the bottomland. Relief influences soil formation

### 3. Site Area Soils

The soils on the right riverbank of Blacksnake River consist of Colo silt loams, poorly drained soils in its low areas. These soils formed in silty alluvium and the slopes ranged from 0 to 2 percent. The surface layer is very dark gray, friable silty clay loam, the subsurface layer is black, friable silty clay loam followed by the subsoil layer consisting of very dark, gray, firm silty clay loam. The permeability is moderate and the surface runoff is slow. The surface layer is friable and can be easily tilled throughout a moderate range in moisture content. The shrink-swell potential is high. A seasonal high water table commonly is at a depth of 1 to 3 feet during extended wet period of time.

## 4. Subsurface Investigation

### 4.1. Drilling and Sampling.

The subsurface investigation for the Blacksnake Creek project was performed by a drill crew from the Kansas City District Corps of Engineers in January 2004. The investigation consisted of performing a total of 6 (six) borings. The boring locations plan view and the strip logs of the exploratory borings are shown in Enclosure 1.

The borings AD-1 and AD-2 were drilled 15 feet deep in the proposed borrow area using a CME 750 ATV drill rig. A 3-¼ inch I.D. inner split barrel sampler was used to obtain sack and jar samples. Alternating layers of silts and clays with interspersed fine sands were encountered throughout both borings to a depth of 15 feet.

Borings ADU-3 and AD-4 were drilled at the location of the proposed levee, along the Blacksnake Creek to a depth between 15 and 16 feet. Boring ADU-3 was drilled with a CME 750 All Terrain Vehicle (ATV) drill rig using 3 ¾ inch I.D. hollow stem augers to advance the hole and a 1-⅜ inch split spoon for sampling. Boring AD-4 was drilled using a Failing 1500 drill rig using a 5 inch box auger to advance the hole and a 1-⅜ inch split spoon for sampling. Undisturbed samples were collected from the boring ADU-3. Alternating layers of silts and clays with interspersed fine sands were encountered throughout both borings to a depth of 15 feet. Limestone cobbles, cement debris and bricks were seen at the surface near the location of ADU-3, but no construction debris was encountered during drilling.

Borings ADU-5 and ADU-6 were drilled at the location of the proposed Kern Road raise. Boring ADU-5, located on the southeast side of the project site near the concrete channel enclosure north of Karnes road, was drilled to 55.5 feet. Continuous undisturbed samples were taken to 49 feet and sampling then continued with S.P.T. every 2.5 feet to 55.5 feet where bedrock was encountered. At 50.5 feet, augering was halted and the drilling method switched to mud rotary and a tricone bit. Clays and silts with interspersed fine sand and a 1.3-foot thick sand with clay was encountered to 51.5 feet. A 0.6-foot thick layer of gravel was found above the bedrock. The bedrock encountered at 54.6 feet was a mudstone or possibly highly weathered shale. One jar sample was obtained from the bedrock. Boring ADU-6 is located closest to Karnes road on the southeast side of the project area. The boring was drilled to the top of rock at 25.8 feet. Clayey silts with fine sand interspersed were encountered to a depth of 24.4 feet. Beginning at 24.4 feet to the top of rock at 25.8 feet a gravel layer was encountered.

Sand was encountered in only one of the boring locations. At a depth of 6.0 feet in boring ADU- an 1.3-foot thick sand layer was logged and sampled. A layer of clayey-gravelly sand 8.4 feet thick was encountered starting at 46.2 feet. A 1.4 feet thick layer of gravelly fat clay was encountered in ADU-6 starting at a depth of 24.4 feet.

The borings were backfilled with neat cement grout to within 3 feet of the ground surface and topped with cuttings from the boreholes. The grout mix consisted of approximately 7.5 gallons of water to each bag of cement. The grout was tremied through drill pipe to the bottom of the boring.

Boring information are summarized in Table 1:

**Table 1 – Borings General Information**

| Boring No. | Investigative Purpose | Total Depth (feet) | No. of Undisturbed Samples | No. of S.P.T. samples | No. of Sack Samples | Water Level (feet) | Sand or gravel encountered (ft) |
|---|---|---|---|---|---|---|---|
| AD-1 | Borrow | 15.0 | | | 3 | None | |
| AD-2 | Borrow | 15.0 | | | 3 | None | |
| ADU-3 | Levee | 15.5 | 6 | | | None | |
| AD-4 | Levee | 16.0 | | 8 | | 5.3 | |
| ADU-5 | Road Raise | 55.5 | 18 | 3 | | 14.2 | 6.0-7.3, 46.2-54.6 |
| ADU-6 | Road Raise | 25.8 | 10 | 1 | | 18.2 | 24.4-25.8 |

4.2. Groundwater.

Groundwater was encountered in 3 of the 6 borings. Water levels were taken at the start of the second day of drilling the 2 deeper borings ADU-5 and ADU-6. Water level in AD-4 was taken 2 hours after drilling was completed.

4.3. Undisturbed Sampling.

Thirty-four (34) undisturbed samples were collected from 3 borings (ADU 3, ADU-5 and ADU-6). Each tube sample was extruded in the field and waxed after torvane strength determinations were run on the top, bottom, and midpoint and a jar sample retained from the base of the sample. Laboratory testing on the undisturbed samples consisted of unconsolidated-undrained triaxial compression tests (Q tests), consolidated undrained triaxial compression tests with pore pressure measurement (R-bar tests), consolidation tests and unconfined compression test. The tests were done on samples collected from borings ADU-3, ADU-5, and ADU-6 to determine the geotechnical characteristics of the foundation soils for the road raise and the levee construction.

4.4. Disturbed Sampling.

Sixty (60) jar samples were collected from all borings. Jar samples were obtained of representative materials encountered between two undisturbed samples, from SPT tests, and at the base of each undisturbed sample. Laboratory testing on the jar samples consisted of determination of the moisture content and Atterberg limits. The jars were grouped based on the visual classifications and Atterberg Limits done on representative sample of each group. The soils will be classified based on Atterberg Limits and visual classification.

4.5. Sac Samples.

Six (6) sacs were collected from the proposed borrow areas. Standard Proctor compaction tests conform ASTM D-698 were performed on the selected sac samples. Unconsolidated-undrained triaxial compression tests (Q tests), consolidated undrained triaxial compression tests with pore pressure measurement (R-bar tests) and were performed on remolded samples compacted to 95% of density as obtained from the Standard Proctor tests.

4.6. Laboratory Testing.

4.6.1. General.

Selected samples of material obtained during the field exploration were tested to determine the engineering and physical properties of the foundation soils and the borrow materials. Laboratory tests were performed by Geotechnology, Inc., Kansas City. The laboratory testing included

Atterberg Limits, natural moisture contents, shear strength parameters of undisturbed soil samples based on unconfined compression tests, unconsolidated undrained (UU) Q triaxial compression tests, and consolidated undrained (CU) R-BAR triaxial compression tests with pore pressure measurements, and consolidation tests. Standard Proctor tests were performed on sac samples collected from the proposed borrow areas. A summary of laboratory testing results is shown in Table 2 "SUMMARY OF LABORATORY TESTING RESULTS".

Table 2 - SUMMARY OF LABORATORY TESTING RESULTS

| Boring # | Depth (ft) | Sample # | Classif. Goup # | LABORATORY TEST | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | Moisture Content % | Visual Classif. | Atterberg Limits | | | | Unconfined Compression Strength (psf) | Triaxial | | | | | Standard Proctor | |
| | | | | | | Liquid Limit (LL) | Plastic Limit (PL) | Plasticity Index (PI) | Liquidity Index (LI) | | Q Test c (psf) | R-Bar Test | | | | $\gamma_{dmax}$ (pcf) | Optimum Moisture Content, % |
| | | | | | | | | | | | | Total Strength | | Effective Strength | | | |
| | | | | | | | | | | | | $\phi$ ($^0$) | c (psf) | $\phi$ ($^0$) | c (psf) | | |
| AD-1 | 0-2.0 | J-1 | 6 | 25.4 | CL | | | | | | | | | | | | |
| | 2.0-4.0 | S-1 | | 16.3 | CL | 37 | 22 | 15 | -38 | | 3080 | 11.5 | 1800 | 34.7 | 0 | 107 | 16 |
| | 2.0-4.0 | J-2 | 6 | 19.9 | CL | | | | | | | | | | | | |
| | 4.0-5.0 | J-3 | 6 | 30.7 | CL | | | | | | | | | | | | |
| | 5.0-8.0 | S-2 | | | CL | 32 | 21 | 11 | | | | | | | | | |
| | 5.0-8.0 | J-4 | 9 | 26.3 | CL | | | | | | | | | | | | |
| | 8.0-10.0 | J-5 | 9 | 27.7 | CL | 37 | 23 | 14 | 34 | | | | | | | | |
| | 10.0-12.0 | J-6 | 1 | 27.9 | CL | | | | | | | | | | | | |
| | 12.0-15.0 | J-7 | 1 | 29.0 | CL | | | | | | | | | | | | |
| AD-2 | 0.0-2.5 | J-1 | 6 | 26.2 | CL | 36 | 21 | 15 | 35 | | | | | | | | |
| | 2.5-5.0 | J-2 | 5 | 24.3 | CL | 35 | 22 | 13 | 18 | | | | | | | | |
| | 2.5-5.0 | S-2 | | | CL | 32 | 22 | 10 | | | | | | | | | |
| | 5.0-7.5 | J-3 | 5 | 24.2 | CL | | | | | | | | | | | | |
| | 7.5-10.0 | J-4 | 6 | 28.3 | CL | | | | | | | | | | | | |
| | 10.0-12.5 | J-5 | 6 | 24.4 | CL | | | | | | | | | | | | |
| | 12.5-15.0 | J-6 | 6 | 27.7 | CL | | | | | | | | | | | | |
| ADU-3 | 2.9-3.0 | J-1 | 5 | 21.6 | CL | | | | | | | | | | | | |
| | 1.0-3.0 | WAX-1 | | 21.7 | CL | 38 | 27 | 11 | -48 | | 1000 | 13.3 | 1000 | 32 | 0 | | |
| | 5.4-5.5 | J-2 | 6 | 20.1 | CL | | | | | | | | | | | | |
| | 7.9-6.00 | J-3 | 6 | 23.5 | CL | | | | | | | | | | | | |
| | 10.4-10.5 | J-4 | 1 | 27.8 | CL | | | | | | | | | | | | |
| | 12.9-13.0 | J-5 | 1 | 27.9 | CL | | | | | | | | | | | | |
| | 15.4-15.5 | J-6 | 1 | 29.1 | CL | | | | | | | | | | | | |
| AD-4 | 0.5-1.0 | J-1 | 5 | 24.6 | CL | | | | | | | | | | | | |
| | 2.5-2.8 | J-2 | 5 | 15.1 | CL | | | | | | | | | | | | |
| | 4.5-5.7 | J-3 | 3 | 25.0 | CL | | | | | | | | | | | | |

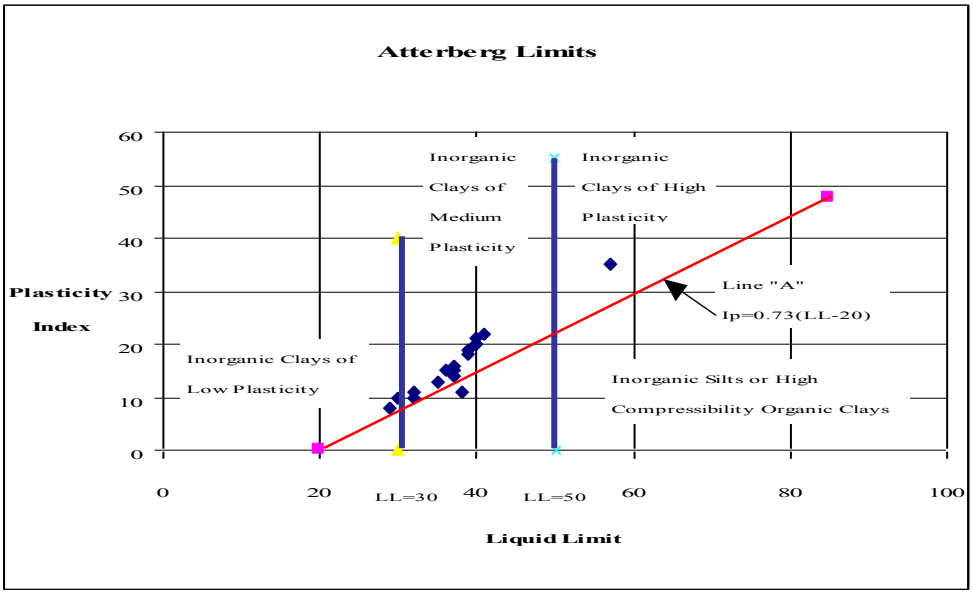
| Boring # | Depth (ft) | Sample # | Classif. Goup # | LABORATORY TEST | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | Moisture Content % | Visual Classif. | Atterberg Limits | | | | Unconfined Compression Strength (psf) | Triaxial | | | | | Standard Proctor | |
| | | | | | | Liquid Limit (LL) | Plastic Limit (PL) | Plasticity Index (PI) | Liquidity Index (LI) | | Q Test c (psf) | R-Bar Test | | | | $\gamma_{dmax}$ (pcf) | Optimum Moisture Content, % |
| | | | | | | | | | | | | Total Strength | | Effective Strength | | | |
| | | | | | | | | | | | | $\phi$ ($^0$) | c (psf) | $\phi$ ($^0$) | c (psf) | | |
| | 6.5-8.0 | J-4 | 6 | 26.6 | CL | | | | | | | | | | | | |
| | 8.5-10.0 | J-5 | 1 | 29.0 | CL | | | | | | | | | | | | |
| | 10.5-12.0 | J-6 | 1 | 27.0 | CL | | | | | | | | | | | | |
| | 12.5-14.0 | J-7 | 10 | 28.5 | CL | | | | | | | | | | | | |
| | 14.5-16.0 | J-8 | 10 | 29.9 | CL | | | | | | | | | | | | |
| ADU-5 | 2.0-3.5 | WAX-1 | | 20.1 | CL | 41 | 19 | 22 | -2 | 3920 | | | | | | | |
| | 3.5-3.6 | J-1 | 5 | 18.6 | CL | | | | | | | | | | | | |
| | 4.5-6.5 | WAX-2 | | 29.6 | CL | 40 | 20 | 20 | 18 | | 800 | 10 | 780 | 8.5 | 960 | | |
| | 6.4-6.5 | J-2 | 5 | 23.6 | CL | | | | | | | | | | | | |
| | 7.0-8.9 | WAX-3 | | 20.5 | CL | 40 | 19 | 21 | 14 | 2160 | | | | | | | |
| | 8.9-9.0 | J-3 | 1 | 21.9 | CL | 40 | 20 | 20 | 17 | | | | | | | | |
| | 9.5-11.4 | WAX-4 | | 23.3 | CL | 39 | 21 | 18 | 28 | 1940 | | | | | | | |
| | 11.4-11.5 | J-4 | 4 | 26.0 | CH | 57 | 22 | 35 | 29 | | | | | | | | |
| | 13.9-14.0 | J5 | 6 | 32.0 | CL | | | | | | | | | | | | |
| | 16.4-16.5 | J-6 | 6 | 33.2 | CL | | | | | | | | | | | | |
| | 18.9-19.0 | J-7 | 6 | 32.6 | CL | | | | | | | | | | | | |
| | 21.4-21.5 | J-8 | 1 | 29.6 | CL | | | | | | | | | | | | |
| | 23.9-24.0 | J-9 | 10 | 27.1 | CL | | | | | | | | | | | | |
| | 26.4-25.0 | J-10 | 10 | 28.1 | CL | | | | | | | | | | | | |
| | 28.9-29.0 | J-11 | 10 | 29.9 | CL | | | | | | | | | | | | |
| | 31.4-35.0 | J-12 | 10 | 28.3 | CL | | | | | | | | | | | | |
| | 33.9-34.0 | J-13 | 1 | 26.2 | CL | | | | | | | | | | | | |
| | 36.4-36.5 | J-14 | 10 | 25.7 | CL | | | | | | | | | | | | |
| | 38.9-39.0 | J-15 | 10 | 25.8 | CL | | | | | | | | | | | | |
| | 41.4-41.5 | J-16 | 10 | 23.4 | CL | | | | | | | | | | | | |
| | 43.9-44.0 | J-17 | 2 | 18.7 | CL | | Non-plastic | | | | | | | | | | |
| | 46.4-46.5 | J-18 | 2 | 17.0 | CL | | | | | | | | | | | | |
| | 49.0-49.5 | J-19 | 3 | 18.7 | CL | | | | | | | | | | | | |
| | 51.5-52.4 | J-20 | 2 | 20.3 | CL | | | | | | | | | | | | |

| Boring # | Depth (ft) | Sample # | Classif. Goup # | LABORATORY TEST | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | Moisture Content % | Visual Classif. | Atterberg Limits | | | | Unconfined Compression Strength (psf) | Triaxial | | | | | Standard Proctor | |
| | | | | | | Liquid Limit (LL) | Plastic Limit (PL) | Plasticity Index (PI) | Liquidity Index (LI) | | Q Test c (psf) | R-Bar Test | | | | $\gamma_{dmax}$ (pcf) | Optimum Moisture Content, % |
| | | | | | | | | | | | | Total Strength | | Effective Strength | | | |
| | | | | | | | | | | | | $\phi$ ($^0$) | c (psf) | $\phi$ ($^0$) | c (psf) | | |
| | 54.0-54.6 | J-21 | 2 | 13.7 | CL | | | | | | | | | | | | |
| | 54.6-55.5 | J-22 | 4 | 21.7 | CH | | | | | | | | | | | | |
| ADU-6 | 0.5-2.4 | WAX-1 | | 19.4 | CL | 40 | 20 | 20 | 19 | 3900 | | | | | | | |
| | 2.4-2.5 | J-1 | 6 | 23.8 | CL | | | | | | | | | | | | |
| | 4.9-5.0 | J-2 | 5 | 20.6 | CL | | | | | | | | | | | | |
| | 5.5-7.4 | WAX-3 | | 23.0 | CL | 39 | 20 | 19 | 16 | 1780 | | | | | | | |
| | 7.4-7.5 | J-3 | 10 | 23.0 | CL | | | | | | | | | | | | |
| | 9.9-10.0 | J-4 | 8 | 21.2 | CL | 30 | 20 | 10 | 29 | | | | | | | | |
| | 12.4-12.5 | J-5 | 10 | 22.9 | CL | | | | | | | | | | | | |
| | 13.0-14.9 | WAX-6 | | 23.2 | CL | 29 | 21 | 8 | 2 | | | | | | | | |
| | 14.9-15.0 | J-6 | 10 | 21.2 | CL | | | | | | | | | | | | |
| | 17.4-17.5 | J-7 | 10 | 22.5 | CL | 37 | 21 | 16 | -6 | | | | | | | | |
| | 19.9-20.0 | J-8 | 10 | 20.1 | CL | | | | | | | | | | | | |
| ADU-6 | 22.4-22.5 | J-9 | 10 | 21.5 | CL | | | | | | | | | | | | |
| | 24.9-25.0 | J-10 | 10 | 22.3 | CL | | | | | | | | | | | | |

4.6.2. Soil Classification Tests.

In an attempt to characterize the site and to classify the soils, moisture content and Atterberg limits tests were conducted. Overburden clay material was classified in accordance with ASTM D 2487 as lean clay (CL), except one sample that was determined to be fat clay. The results of the 68 natural moisture content tests performed on disturbed and undisturbed samples show the moisture content ranges between 13.7 and 33.2 percent with the average of 24.4%. The results of the Atterberg Limits tests performed on 18 samples of clay material are presented in Table 2 and on Casagrande's plasticity chart in Figure 1. As shown in Table 2, the soils encountered in the foundation consist of lean clays, with low plasticity index and high water content, except one sample within a thin layer of fat clay with high plasticity encountered in Boring ADU-5 at a depth of 11.5 feet from the surface. The Liquid Limit (LL) varies between 29 and 41 percent with one exception where the LL was found to be 57. The Plastic Limit (PL) varies between19 and 27 with an average of 21.

**Figure 1 – Atterberg Limits**




4.6.3. Consolidation Tests.

One consolidation test was performed on undisturbed clay samples collected in Boring ADU-5 between 4.5 and 6.5 feet below existing grade. A summary of consolidation test results is shown in Table 3.

**TABLE C-8 - CONSOLIDATION TESTS SUMMARY**

| Boring No. | Sample No. | Depth feet | Elev. ft | Moist. % | LL | PL | PI | LI | $C_c$ | $C_r$ | $e_0$ | $P_p$ tsf |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ADU-5 | Wax-2 | 4.5-6.5 | 890 | 29.6 | 40 | 20 | 20 | 18 | 0.138 | 0.013 | 0.9222 | 1.6 |

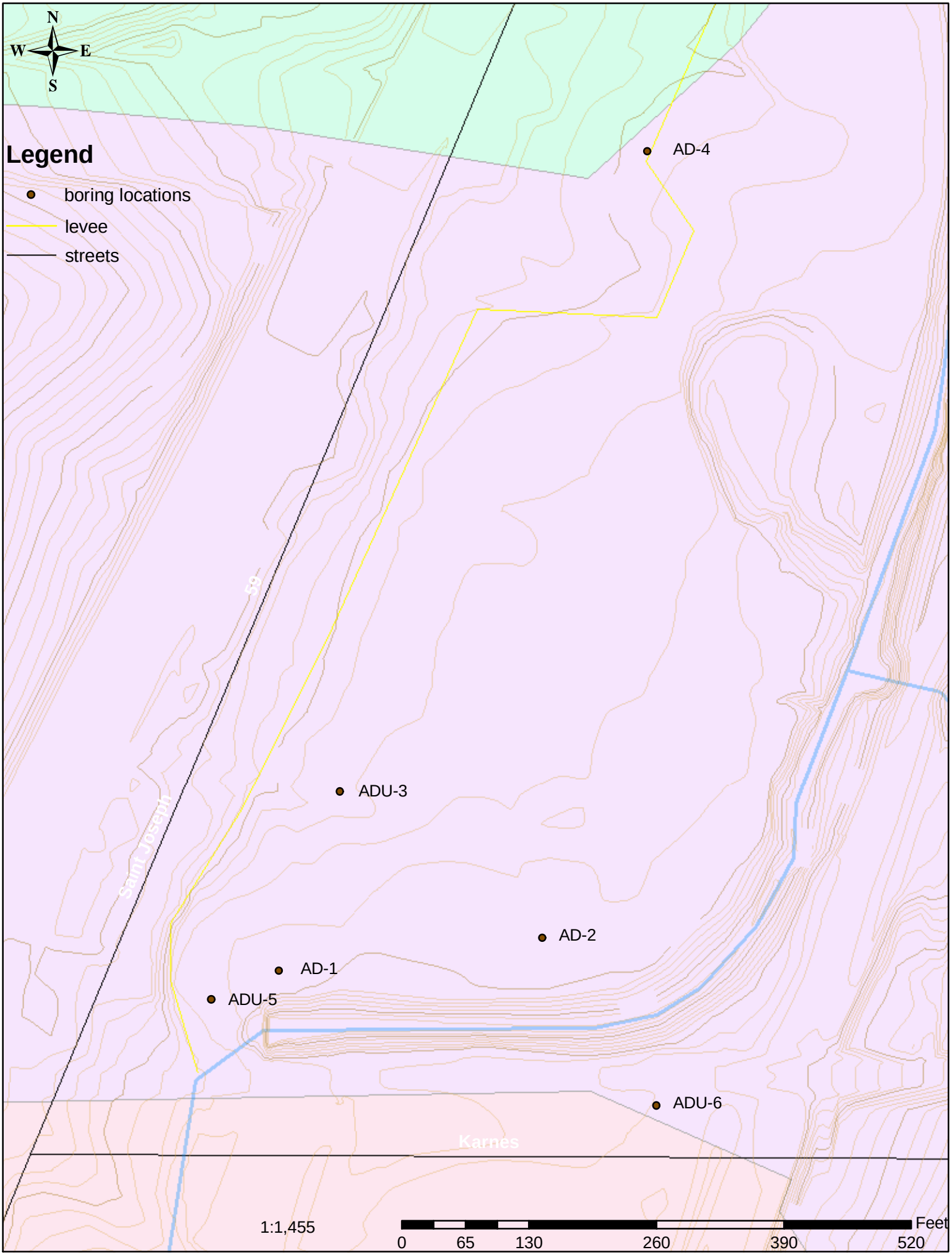
#### 4.6.4. Unconfined Compression Tests

A total of five (5) unconfined compression tests were performed on undisturbed samples collected form the borings ADU-5 and ADU-6 at depth varying between 2 and 15 feet below the existing surface. The results of the unconfined compression tests are shown on Table 2. The soil shear strength varies between 1780 psf and 3900 psf, with strength decreasing with the depth.

#### 4.6.5. Triaxial Compression Tests.

Unconsolidated undrained (UU) Q triaxial compression tests. Three (3) unconsolidated undrained (UU) Q triaxial compression tests were performed, one of them on remolded sample from the borrow area and 2 tests on undisturbed samples collected from borings ADU-3 and ADU-5, at a depth of 1 to 3 feet and 4.5 to 6.5 feet respectively. The remolded sample collected from the proposed borrow area was compacted at approximately 95 percent Standard Proctor maximum density, determined by ASTM D-698-91, adjusted to a range between 2 and 3 percent above optimum moisture content. A series consists of three Q tests conducted on three specimens trimmed from a single Shelby tube sample. The test series were conducted at confining pressures of 1 tsf. Results of the Q triaxial compression tests are provided in Table 2.

Consolidate Undrained (CU) triaxial tests (R-Bar). Two (2) test series of consolidated undrained (CU) triaxial tests with pore water pressure measurements were performed on undisturbed samples collected from SADU-3 and ADU-5 at depths up to 6.5feet below the existing ground surface. One CU test was performed on remolded sample, collected from the proposed borrow area, compacted at approximately 95 percent Standard Proctor maximum density, determined by ASTM D-698-91, adjusted to a range of between 2 to 3 percent above optimum moisture content. A consolidated undrained test series consists of three tests conducted on three specimens trimmed from a single Shelby tube sample, saturated and consolidated at different confining pressures to define the strength envelope. The test series were conducted at confining pressures of 0.5, 1, and 2 tsf. Shear strength in terms of effective strength (S strength) were established from the CU tests by measuring the induced pore water pressures. Results of the CU triaxial tests in terms of effective and total strength parameters are provided in Table 2.


N
W
E
S
Legend
boring locations
levee
streets
AD-4
59
ADU-3
Saint Joseph
AD-2
AD-1
ADU-5
ADU-6
Karnes
1:1,455
0
65
130
260
390
520
Feet

# LOG OF BORING AD-1



US Army Corps of Engineers ®

Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Blacksnake Creek
PROJECT: Road Raise and Levee Investigation
LOCATION: Saint Joseph, MO
COORDINATES: N 1323043.27, E 2692161.78
DATE(S) DRILLED: 3/1/04 - 3/1/04

FIELD DATA

DRILLING METHOD(S):
CME 750, 3 3/4" Hollow Stem Auger

GROUNDWATER INFORMATION:
Not encountered

SURFACE ELEVATION: 0

LABORATORY DATA

| DEPTH (FT) | SOIL SYMBOL | BREAKS: bb or mb | SAMPLES | N: BLOWS/FT P: TONS/SQ FT T: TORVANE KG/CM SQ R: % RQD: % | DESCRIPTION OF STRATUM | USCS SYMBOL | ATTERBERG LIMITS: LIQUID LIMIT LL | ATTERBERG LIMITS: PLASTIC LIMIT PL | MOISTURE CONTENT (%) | UNCONFINED COMPRESSIVE STRENGTH (TONS/SQ FT) | MINUS NO. 200 SIEVE (%) | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | FAT CLAY<br>MEDIUM<br>DAMP TO MOIST<br>BROWN TO DARK BROWN<br>-rootlets<br>-contains sparse, occasional very fine sand, < 15% | | | | | | | | |
| 1 | | | | | | | | | | | | | |
| 2 | | | | | | | | | | | | | |
| 3 | | | | | | | | | | | | | |
| 4 | | | | | | | | | | | | | |
| 5 | | | | | CLAYEY SILT<br>MEDIUM<br>DAMP<br>BROWN TO DARK BROWN<br>-contains sparse occasional very fine sand, < 15% | | | | | | | | |
| 6 | | | | | | | | | | | | | |
| 7 | | | | | | | | | | | | | |
| 8 | | | | | | | | | | | | | |
| 9 | | | | | | | | | | | | | |
| 10 | | | | | FAT CLAY<br>MEDIUM<br>DAMP TO MOIST<br>BROWN & GREENISH GRAY<br>-contains sparse, occasional very fine sand, < 15% | | | | | | | | |
| 11 | | | | | | | | | | | | | |
| 12 | | | | | | | | | | | | | |
| 13 | | | | | | | | | | | | | |
| 14 | | | | | | | | | | | | | |
| 15 | | | | | Bottom of Hole | | | | | | | | |

N - STANDARD PENETRATION TEST RESISTANCE
P - POCKET PENETROMETER RESISTANCE
T - TORVANE EQUALLY SPACED ALONG SAMPLE
R - ROCK CORE RECOVERY
RQD - ROCK QUALITY DESIGNATION

REMARKS:
Backfill with Portland cement dry bagged to 3.0' BGS and topped with spoils.
Coordinates Trimble Hand GPS US State Plane MO west 2403 NAD 83 feet.

# LOG OF BORING AD-2



US Army Corps of Engineers ®

Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Blacksnake Creek
PROJECT: Road Raise and Levee Investigation
LOCATION: Saint Joseph, MO
COORDINATES: N 1323077.02, E 2692430.74
DATE(S) DRILLED: 3/1/04 - 3/1/04

FIELD DATA

DRILLING METHOD(S):
CME 750, 3 3/4" Hollow Stem Auger

GROUNDWATER INFORMATION:
Not encountered

SURFACE ELEVATION: 0

LABORATORY DATA

| DEPTH (FT) | SOIL SYMBOL | BREAKS: bb or mb | SAMPLES | N: BLOWS/FT P: TONS/SQ FT T: TORVANE KG/CM SQ R: % RQD: % | DESCRIPTION OF STRATUM | USCS SYMBOL | ATTERBERG LIMITS: LIQUID LIMIT LL | ATTERBERG LIMITS: PLASTIC LIMIT PL | MOISTURE CONTENT (%) | UNCONFINED COMPRESSIVE STRENGTH (TONS/SQ FT) | MINUS NO. 200 SIEVE (%) | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | FAT CLAY<br>DAMP TO MOIST<br>MEDIUM<br>BROWN<br>-silty<br>-rootlets<br>-contains <15 % very fine sand | | | | | | | | |
| | | | | | FAT CLAY<br>MEDIUM<br>DAMP<br>BROWN AND DARK GRAY<br><contains 15% very fine sand occasionally | | | | | | | | |
| | | | | | FAT CLAY<br>MEDIUM<br>MOIST<br>BROWN AND GREENISH GRAY<br><contains 15% very fine sand occasionally | | | | | | | | |
| | | | | | BOH | | | | | | | | |

N - STANDARD PENETRATION TEST RESISTANCE
P - POCKET PENETROMETER RESISTANCE
T - TORVANE EQUALLY SPACED ALONG SAMPLE
R - ROCK CORE RECOVERY
RQD - ROCK QUALITY DESIGNATION

REMARKS:
Backfill with 2 bags Portland cement dry bagged and topped with spoils.
Coordinates Trimble Hand GPS US State Plane MO west 2403 NAD 83 feet.

# LOG OF BORING AD-4



US Army Corps of Engineers ®

Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Blacksnake Creek
PROJECT: Road Raise and Levee Investigation
LOCATION: Saint Joseph, MO
COORDINATES: N 1323880.13, E 2692538.1
DATE(S) DRILLED: 2/26/04 - 2/26/04

FIELD DATA

DRILLING METHOD(S):
Failing 1500, 5 inch box auger, 1 3/8" standard split spoon

GROUNDWATER INFORMATION:
2 hours after drilling

SURFACE ELEVATION: 0

LABORATORY DATA

| DEPTH (FT) | SOIL SYMBOL | BREAKS: bb or mb | SAMPLES | N: BLOWS/FT P: TONS/SQ FT T: TORVANE KG/CM SQ R: % RQD: % | DESCRIPTION OF STRATUM | USCS SYMBOL | ATTERBERG LIMITS: LIQUID LIMIT LL | ATTERBERG LIMITS: PLASTIC LIMIT PL | MOISTURE CONTENT (%) | UNCONFINED COMPRESSIVE STRENGTH (TONS/SQ FT) | MINUS NO. 200 SIEVE (%) | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 – 2 | | | X | N = 8 | FAT CLAY<br>SOFT<br>DAMP<br>BROWN TO DARK BROWN<br>-rootlets | | | | | | | | |
| 3 – 4 | | | X | N = 21 | CLAYEY SILT<br>STIFF<br>DAMP<br>BROWN<br>< 15% very fine sand<br>-sparse occasional rootlets | | | | | | | | |
| 5 – 6 | | | X | N = 14 | FAT CLAY<br>MEDIUM<br>DAMP<br>BROWN<br>- contains < 15 % very fine sand<br>- silty | | | | | | | | |
| 7 – 8 | | | | | FAT CLAY<br>SOFT<br>DAMP TO MOIST<br>BROWN<br>- contains < 15% very fine sand | | | | | | | | |
| 9 – 10 | | | X | N = 5 | | | | | | | | | |
| 11 – 12 | | | X | N = 6 | | | | | | | | | |
| 13 – 14 | | | X | N = 6 | | | | | | | | | |
| 15 – 16 | | | X | N = 3 | | | | | | | | | |
| 16 | | | | | BOH | | | | | | | | |

N - STANDARD PENETRATION TEST RESISTANCE
P - POCKET PENETROMETER RESISTANCE
T - TORVANE EQUALLY SPACED ALONG SAMPLE
R - ROCK CORE RECOVERY
RQD - ROCK QUALITY DESIGNATION

REMARKS:
Backfill with grout tremied to 2.0' BGS with 2 bags of portland cement, topped with spoils to ground surface
Coordinates Trimble Hand GPS US State Plane MO west 2403 NAD 83 feet.

# LOG OF BORING ADU-3



US Army Corps of Engineers
Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Blacksnake Creek
PROJECT: Road Raise and Levee Investigation
LOCATION: Saint Joseph, MO
COORDINATES: N 1323226.77, E 2692224.11
DATE(S) DRILLED: 3/1/04 - 3/1/04

FIELD DATA

DRILLING METHOD(S):
CME 750, 3 3/4" Hollow Stem Auger, 5" Shelby Tube

GROUNDWATER INFORMATION:
Not encountered

SURFACE ELEVATION: 0

LABORATORY DATA

| DEPTH (FT) | SOIL SYMBOL | BREAKS: bb or mb | SAMPLES | N: BLOWS/FT P: TONS/SQ FT T: TORVANE KG/CM SQ R: % RQD: % | DESCRIPTION OF STRATUM | USCS SYMBOL | ATTERBERG LIMITS: LIQUID LIMIT LL | ATTERBERG LIMITS: PLASTIC LIMIT PL | MOISTURE CONTENT (%) | UNCONFINED COMPRESSIVE STRENGTH (TONS/SQ FT) | MINUS NO. 200 SIEVE (%) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | FAT CLAY<br>MEDIUM<br>DAMP TO MOIST<br>BROWN<br>-rootlets | | | | | | |
| 1 | | | | | | | | | | | |
| 2 | | | | T = .4, .4, .4 | FAT CLAY<br>STIFF<br>DAMP<br>BROWN TO REDDISH BROWN<br>-contains sparse, occasional very fine sand, <15% | | | | | | |
| 3 | | | | | | | | | | | |
| 4 | | | | T = .35, .9, >1 | | | | | | | |
| 5 | | | | | | | | | | | |
| 6 | | | | | | | | | | | |
| 7 | | | | T = .9, .8, .85 | | | | | | | |
| 8 | | | | | | | | | | | |
| 9 | | | | T = .7, .65, .7 | | | | | | | |
| 10 | | | | | | | | | | | |
| 11 | | | | | FAT CLAY<br>STIFF TO MEDIUM<br>DAMP<br>BROWN<br>-contains sparse, occasional very fine sand, <15% | | | | | | |
| 12 | | | | T = .65, .43, .5 | | | | | | | |
| 13 | | | | | FAT CLAY<br>STIFF TO MEDIUM<br>DAMP<br>BROWN<br>-contains sparse, occasional very fine sand, <15% | | | | | | |
| 14 | | | | T = .65, .6, .5 | | | | | | | |
| 15 | | | | | | | | | | | |
| | | | | | BOTTOM OF HOLE | | | | | | |

N - STANDARD PENETRATION TEST RESISTANCE
P - POCKET PENETROMETER RESISTANCE
T - TORVANE EQUALLY SPACED ALONG SAMPLE
R - ROCK CORE RECOVERY
RQD - ROCK QUALITY DESIGNATION

REMARKS:
Backfill with 2 bags Portland cement dry bagged and topped with spoils.
Coordinates Trimble Hand GPS US State Plane MO west 2403 NAD 83 feet.

# LOG OF BORING ADU-5



US Army Corps of Engineers
Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Blacksnake Creek
PROJECT: Road Raise and Levee Investigation
LOCATION: Saint Joseph, MO
COORDINATES: N 1323014.45, E 2692092.69
DATE(S) DRILLED: 2/24/04 - 2/25/04

DRILLING METHOD(S):
Failing 1500, 5 inch box auger, 5" shelby tube, 1 3/8" standard split spoon

GROUNDWATER INFORMATION:
overnight, during drilling 2/25/04 0700

SURFACE ELEVATION: 0

FIELD DATA: DEPTH (FT) | SOIL SYMBOL | BREAKS: bb or mb | SAMPLES | N: BLOWS/FT P: TONS/SQ FT T: TORVANE KG/CM SQ R: % RQD: %

LABORATORY DATA: USCS SYMBOL | ATTERBERG LIMITS: LIQUID LIMIT (LL), PLASTIC LIMIT (PL) | MOISTURE CONTENT (%) | UNCONFINED COMPRESSIVE STRENGTH (TONS/SQ FT) | MINUS NO. 200 SIEVE (%)

| DEPTH (FT) | DESCRIPTION OF STRATUM | N: BLOWS/FT P: TONS/SQ FT T: TORVANE KG/CM SQ R: % RQD: % |
|---|---|---|
| 0–2 | FAT CLAY<br>MEDIUM<br>DAMP TO MOIST<br>BROWN<br>-rootlets | |
| 2–5 | SILTY LEAN CLAY WITH SAND<br>STIFF TO VERY STIFF<br>DAMP<br>BROWN TO DARK BROWN<br>-fill<br>-contains sparse, occasional fine to coarse limestone gravel up to 2" in size <15%<br>-contains very fine to fine sand, <15% | |
| 5–7 | FAT CLAY<br>MEDIUM<br>DAMP<br>BROWN TO LIGHT BROWN<br>-contains occasional very fine sand <15% | T = .3, .25, .25 |
| 7–9 | CLAYEY SAND<br>DENSE TO MEDIUM DENSE<br>DAMP<br>BROWN TO DARK BROWN<br>-contains very fine to fine sand 45% to 55% | T = .55, .25, .75 |
| 9–11 | FAT CLAY<br>STIFF<br>DAMP<br>BROWN TO DARK BROWN<br>-sparse occasional very fine to fine sand <15% | T = .85, .7, .6 |
| 11–13 | FAT CLAY<br>STIFF<br>DAMP<br>BROWN TO LIGHT BROWN<br>-sparse occasional very fine to fine sand <15% | T = .4, .4, .2 |
| 13–15 | SANDY FAT CLAY<br>MEDIUM TO STIFF<br>DAMP TO MOIST<br>DARK GRAY<br>-contains very fine to fine sand 30% to 40%<br>-poorly laminated | T = .4, .35, .2 |
| 15–17 | SANDY FAT CLAY<br>MEDIUM<br>MOIST<br>DARK GRAY<br>-contains very fine to fine sand 30% to 40%<br>-poorly laminated | T = .3, .35, .4 |
| 17–20 | SANDY FAT CLAY<br>MEDIUM TO STIFF<br>DAMP TO MOIST<br>DARK GRAY | |

N - STANDARD PENETRATION TEST RESISTANCE
P - POCKET PENETROMETER RESISTANCE
T - TORVANE EQUALLY SPACED ALONG SAMPLE
R - ROCK CORE RECOVERY
RQD - ROCK QUALITY DESIGNATION

REMARKS:
Backfill with grout tremied to 2.0' BGS with 8 bags of portland cement, topped with spoils
Coordinates Trimble Hand GPS US State Plane MO west 2403 NAD 83 feet.

# LOG OF BORING ADU-5



US Army Corps of Engineers
Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Blacksnake Creek
PROJECT: Road Raise and Levee Investigation
LOCATION: Saint Joseph, MO
COORDINATES: N 1323014.45, E 2692092.69
DATE(S) DRILLED: 2/24/04 - 2/25/04

DRILLING METHOD(S):
Failing 1500, 5 inch box auger, 5" shelby tube, 1 3/8" standard split spoon

GROUNDWATER INFORMATION:
overnight, during drilling 2/25/04 0700

FIELD DATA: DEPTH (FT) | SOIL SYMBOL | BREAKS: bb or mb | SAMPLES | N: BLOWS/FT P: TONS/SQ FT T: TORVANE KG/CM SQ R: % RQD: %

LABORATORY DATA: USCS SYMBOL | ATTERBERG LIMITS: LIQUID LIMIT (LL), PLASTIC LIMIT (PL) | MOISTURE CONTENT (%) | UNCONFINED COMPRESSIVE STRENGTH (TONS/SQ FT) | MINUS NO. 200 SIEVE (%)

| DEPTH (FT) | DESCRIPTION OF STRATUM | N: BLOWS/FT P: TONS/SQ FT T: TORVANE KG/CM SQ R: % RQD: % |
|---|---|---|
| | -contains very fine to fine sand 30% to 40%<br>-poorly laminated | T = .3, .4 |
| 21 | SANDY FAT CLAY<br>MEDIUM<br>DAMP<br>DARK GRAY<br>-contains very fine to fine sand 25% to 35%<br>-poorly laminated | |
| 22 | | |
| 23 | SILTY LEAN CLAY WITH SAND<br>MEDIUM TO STIFF<br>DAMP<br>DARK GRAY TO DARK GREENISH GRAY<br>-contains 15% to 25% very fine sand | T = .2, .2, .2 |
| 24 | | |
| 25 | -20.4' to 20.5' layer containing very fine to medium sand in matrix<br>-20.8' to 21.1' layer containing very fine to coarse sand in matrix | T = .2, .3, .25 |
| 26 | | |
| 27 | SILTY LEAN CLAY WITH SAND<br>MEDIUM TO STIFF<br>MOIST TO WET<br>BROWN TO LIGHT BROWN<br>-contains 15% to 25% very fine sand | |
| 28 | | T = .3, .3, .3 |
| 29 | | |
| 30 | | T = .2, .15, .15 |
| 31 | | |
| 32 | | |
| 33 | SILTY LEAN CLAY<br>STIFF<br>DAMP TO MOIST<br>BROWN<br>-contains <15% very fine sand | T = .3, .4, .4 |
| 34 | | |
| 35 | | T = .55, .55, .55 |
| 36 | | |
| 37 | | |
| 38 | | T = .6, .6, .6 |
| 39 | SANDY FAT CLAY<br>STIFF | |
| 40 | | |

N - STANDARD PENETRATION TEST RESISTANCE
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T - TORVANE EQUALLY SPACED ALONG SAMPLE
R - ROCK CORE RECOVERY
RQD - ROCK QUALITY DESIGNATION

REMARKS:
Backfill with grout tremied to 2.0' BGS with 8 bags of portland cement, topped with spoils
Coordinates Trimble Hand GPS US State Plane MO west 2403 NAD 83 feet.

# LOG OF BORING ADU-5



US Army Corps of Engineers ®

Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Blacksnake Creek
PROJECT: Road Raise and Levee Investigation
LOCATION: Saint Joseph, MO
COORDINATES: N 1323014.45, E 2692092.69
DATE(S) DRILLED: 2/24/04 - 2/25/04

DRILLING METHOD(S):
Failing 1500, 5 inch box auger, 5" shelby tube, 1 3/8" standard split spoon

GROUNDWATER INFORMATION:
overnight, during drilling 2/25/04 0700

FIELD DATA: DEPTH (FT) | SOIL SYMBOL | BREAKS: bb or mb | SAMPLES | N: BLOWS/FT P: TONS/SQ FT T: TORVANE KG/CM SQ R: % RQD: %

LABORATORY DATA: USCS SYMBOL | ATTERBERG LIMITS: LIQUID LIMIT (LL), PLASTIC LIMIT (PL) | MOISTURE CONTENT (%) | UNCONFINED COMPRESSIVE STRENGTH (TONS/SQ FT) | MINUS NO. 200 SIEVE (%)

| DEPTH (FT) | SAMPLES | DESCRIPTION OF STRATUM |
|---|---|---|
| 40–46 | T = .35, .35, .35<br>T = .25, .4, .25<br>T = .25, .15, .2 | MOIST<br>BROWN<br>-with silt<br>-contains 20% to 30% very fine to fine sand |
| 46–51 | N = 33 | CLAYEY SAND<br>DENSE<br>WET<br>BROWN<br>-contains 50% to 60% very fine to coarse sand |
| 51–53 | N = 30 | CLAYEY SAND<br>MEDIUM<br>SATURATED<br>BROWN<br>-contains 55% to 65% very fine to coarse sand, sparse fine gravel |
| 53–55 | N = 52 | SANDY GRAVEL<br>MEDIUM<br>SATURATED<br>BROWN<br>-contains 40% to 50% fine to coarse gravel up to 1" diameter, poorly sorted<br>-contains 30% to 40% fine to coarse sand, dominately medium to coarse |
| 55 | | MUDSTONE<br>SOFT<br>DENSE TO VERY FINE GRAINED<br>GREENISH GRAY AND ORANGE<br>-poorly laminated<br>BOH |

N - STANDARD PENETRATION TEST RESISTANCE
P - POCKET PENETROMETER RESISTANCE
T - TORVANE EQUALLY SPACED ALONG SAMPLE
R - ROCK CORE RECOVERY
RQD - ROCK QUALITY DESIGNATION

REMARKS:
Backfill with grout tremied to 2.0' BGS with 8 bags of portland cement, topped with spoils
Coordinates Trimble Hand GPS US State Plane MO west 2403 NAD 83 feet.

# LOG OF BORING ADU-6



US Army Corps of Engineers ®

Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Blacksnake Creek
PROJECT: Road Raise and Levee Investigation
LOCATION: Saint Joseph, MO
COORDINATES: N 1322906.04, E 2692547.52
DATE(S) DRILLED: 2/23/04 - 2/24/04

FIELD DATA

DRILLING METHOD(S):
Failing 1500, 5 inch box auger, 5" shelby tube, 1 3/8" standard split spoon

GROUNDWATER INFORMATION:
overnight during drilling 2/24/04 0700

SURFACE ELEVATION: 0

LABORATORY DATA

| DEPTH (FT) | SOIL SYMBOL | BREAKS: bb or mb | SAMPLES | N: BLOWS/FT P: TONS/SQ FT T: TORVANE KG/CM SQ R: % RQD: % | DESCRIPTION OF STRATUM | USCS SYMBOL | ATTERBERG LIMITS: LIQUID LIMIT LL | ATTERBERG LIMITS: PLASTIC LIMIT PL | MOISTURE CONTENT (%) | UNCONFINED COMPRESSIVE STRENGTH (TONS/SQ FT) | MINUS NO. 200 SIEVE (%) | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | | | | | FAT CLAY<br>MEDIUM TO STIFF<br>DAMP<br>BROWN TO DARK BROWN | | | | | | | | |
| 2 | | | | T = .55, .9, .75 | <15% very fine sand<br>-silty | | | | | | | | |
| 3 | | | | | | | | | | | | | |
| 4 | | | | T = .55, .6, .5 | | | | | | | | | |
| 5 | | | | | | | | | | | | | |
| 6 | | | | | | | | | | | | | |
| 7 | | | | T = .55, .55, .7 | FAT CLAY<br>STIFF<br>DAMP<br>BROWN TO LIGHT BROWN | | | | | | | | |
| 8 | | | | | -<15% very fine to fine sand<br>-silty | | | | | | | | |
| 9 | | | | T = .85, .9, .65 | | | | | | | | | |
| 10 | | | | | ELASTIC SILT<br>STIFF<br>DAMP<br>BROWN TO LIGHT BROWN | | | | | | | | |
| 11 | | | | T = .7, .55, .65 | <15% very fine to fine sand<br>-clayey | | | | | | | | |
| 12 | | | | | | | | | | | | | |
| 13 | | | | | FAT CLAY<br>STIFF<br>DAMP<br>BROWN TO LIGHT BROWN | | | | | | | | |
| 14 | | | | T = .4, .35, .55 | | | | | | | | | |
| 15 | | | | | -<15% very fine to fine sand<br>-silty | | | | | | | | |
| 16 | | | | T = .65, .45, .55 | | | | | | | | | |
| 17 | | | | | | | | | | | | | |
| 18 | | | | | | | | | | | | | |
| 19 | | | | T = .45, .55, .6 | FAT CLAY<br>STIFF | | | | | | | | |
| 20 | | | | | | | | | | | | | |

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P - POCKET PENETROMETER RESISTANCE
T - TORVANE EQUALLY SPACED ALONG SAMPLE
R - ROCK CORE RECOVERY
RQD - ROCK QUALITY DESIGNATION

REMARKS:
Backfill with grout tremied to 2.0' BGS with 3 bags of portland cement, topped with spoils to ground surface
Coordinates Trimble Hand GPS US State Plane MO west 2403 NAD 83 feet.

# LOG OF BORING ADU-6



US Army Corps of Engineers

Department of the Army
Kansas City District
Corps of Engineers
700 Federal Building
Kansas City, MO 64106

INSTALLATION: Blacksnake Creek
PROJECT: Road Raise and Levee Investigation
LOCATION: Saint Joseph, MO
COORDINATES: N 1322906.04, E 2692547.52
DATE(S) DRILLED: 2/23/04 - 2/24/04

FIELD DATA

DRILLING METHOD(S):
Failing 1500, 5 inch box auger, 5" shelby tube, 1 3/8" standard split spoon

GROUNDWATER INFORMATION:
overnight during drilling 2/24/04 0700

LABORATORY DATA

| DEPTH (FT) | SOIL SYMBOL | BREAKS: bb or mb | SAMPLES | N: BLOWS/FT P: TONS/SQ FT T: TORVANE KG/CM SQ R: % RQD: % | DESCRIPTION OF STRATUM | USCS SYMBOL | ATTERBERG LIMITS: LIQUID LIMIT LL | ATTERBERG LIMITS: PLASTIC LIMIT PL | MOISTURE CONTENT (%) | UNCONFINED COMPRESSIVE STRENGTH (TONS/SQ FT) | MINUS NO. 200 SIEVE (%) | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | DAMP<br>LIGHT BROWN TO LIGHT REDDISH BROWN<br>-silty | | | | | | | | |
| 21 | | | | T = >1, >1, | >1 | | | | | | | | |
| 22 | | | | | | | | | | | | | |
| 23 | | | | | | | | | | | | | |
| 24 | | | | T = >1, >1, | .6 | | | | | | | | |
| | | | | | GRAVELLY FAT CLAY<br>STIFF<br>DAMP TO MOIST<br>LIGHT BROWN TO LIGHT REDDISH BROWN<br>-silty<br>-contains fine to 30% to 40% coarse limestone gravel and cobbles up to 6" in diameter | | | | | | | | |
| 25 | | | | | | | | | | | | | |
| | | | | N = 50 | | | | | | | | | |
| | | | | | GRAVELLY FAT CLAY<br>STIFF<br>SATURATED<br>LIGHT BROWN TO LIGHT REDDISH BROWN<br>-silty<br>-contains fine to 30% to 40% coarse limestone gravel and cobbles up to 6" in diameter<br>BOH | | | | | | | | |

N - STANDARD PENETRATION TEST RESISTANCE
P - POCKET PENETROMETER RESISTANCE
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REMARKS:
Backfill with grout tremied to 2.0' BGS with 3 bags of portland cement, topped with spoils to ground surface
Coordinates Trimble Hand GPS US State Plane MO west 2403 NAD 83 feet.

GEOTECHNOLOGY, INC.
ENGINEERING AND ENVIRONMENTAL SERVICES
SAINT LOUIS • COLLINSVILLE • KANSAS CITY

August 16, 2004

Ms. Mary Perlea
U.S. Army Corps of Engineers
NWK-EC-GD
601 East 12th Street
Kansas City, MO 64106

RECEIVED
OCT 2 2011
STANLEY CONSULTANTS
MINNEAPOLIS

Reference: Indefinite Delivery Contract No. DACA41-03-D-0003

Dear Ms. Perlea:

Please find attached laboratory testing results for the Blacksnake Creek project under the referenced contract. We appreciate the continued opportunity to provide services to the Corps under this contract. Please feel free to call with any questions.

Very truly yours,

Geotechnology, Inc.

STATE OF MISSOURI
YINGWEI WU
NUMBER
PROFESSIONAL ENGINEER

Yingwei Wu, Ph.D. P.E.
Staff Engineer

YAW:yaw

Copies submitted: (2)

Attachments: Laboratory Testing Results

GEOTECHNOLOGY, INC.
ENGINEERING AND ENVIRONMENTAL SERVICES
ST. LOUIS • COLLINSVILLE • KANSAS CITY

## SUMMARY OF CLASSIFICATIONS TESTS

***Project: Black Snake River***

Note: By visual examination and classification, samples not tested were compared and grouped with typical test samples described below:

| | | |
|---|---|---|
| 1) | Lean Clay (CL) | Brown and dark brown clay<br>(LL - 40, PI - 20) |
| 2) | Sand and Gravel | Brown sand and gravel<br>(LL - N/A, PI – Non-Plastic) |
| 3) | Silt (ML) | Brown to light grey clayey sandy silt<br>(LL - N/A, PI - N/A) |
| 4) | Fat Clay (CH) | Grey to dark yellow clay<br>(LL - 57, PI - 22) |
| 5) | Lean Clay (CL) | Dark brown silty clay<br>(LL - 35, PI - 13) |
| 6) | Lean Clay (CL) | Dark brown, sandy silty clay<br>(LL - 36, PI - 15) |
| 7) | Fat Clay (CH) | Brown clay with sand and gravel<br>(LL - N/A, PI - N/A) |
| 8) | Silty Clay (CL) | Light brown silty clay<br>(LL - 30, PI - 10) |
| 9) | Lean Clay (CL) | Dark brown silty clay, trace sand<br>(LL - 37, PI - 14) |
| 10) | Lean Clay (CL) | Light brown to brown silty clay<br>(LL - 37, PI - 16) |

| BORING NO. | SAMPLE DEPTH (feet) | SAMPLE NUMBER | LABORATORY TESTS | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Classification Group Number | Moisture Content, % | Atterberg Limit | | | $\gamma_t$ (pcf) | Unconfined Compression | Standard Proctor | |
| | | | | | Liquid Limit | Plastic Limit | Plasticity Index | | qu, (tsf) | $\gamma_{dmax}$ (pcf) | Optimum Moisture Content, % |
| | | | | | | | | | | | |
| AD-1 | 0-2.0 | Jar-1 | 6 | 25.4 | | | | | | | |
| | 2.0-4.0 | Sack-1 | | N/T* | 37 | 22 | 15 | | | 107.0 | 16.0 |
| | 2.0-4.0 | Jar-2 | 6 | 19.9 | | | | | | | |
| | 4.0-5.0 | Jar-3 | 6 | 30.7 | | | | | | | |
| | 5.0-8.0 | Sack-2 | | N/T* | 32 | 21 | 11 | | | Not Enough Materials | |
| | 5.0-8.0 | Jar-4 | 9 | 26.3 | | | | | | | |
| | 8.0-10.0 | Jar-5 | 9 | 27.7 | 37 | 23 | 14 | | | | |
| | 10.0-12.0 | Jar-6 | 1 | 27.9 | | | | | | | |
| | 12.0-15.0 | Jar-7 | 1 | 29.0 | | | | | | | |
| AD-2 | 0.0-2.5 | Jar-1 | 6 | 26.2 | 36 | 21 | 15 | | | | |
| | 2.5-5.0 | Jar-2 | 5 | 24.3 | 35 | 22 | 13 | | | | |
| | 2.5-5.0 | Sack-2 | | N/T* | 32 | 22 | 10 | | | 105.5 | 16.5 |
| | 5.0-7.5 | Jar-3 | 5 | 24.2 | | | | | | | |
| | 7.5-10.0 | Jar-4 | 6 | 28.3 | | | | | | | |
| | 10.0-12.5 | Jar-5 | 6 | 24.4 | | | | | | | |
| | 12.5-15.0 | Jar-6 | 6 | 27.7 | | | | | | | |
| ADU-3 | 2.9-3.0 | Jar-1 | 5 | 21.6 | | | | | | | |
| | 1.0-3.0 | WAX-1 | | 21.7 | 38 | 27 | 11 | | | | |
| | 5.4-5.5 | Jar-2 | 6 | 20.1 | | | | | | | |
| | 6.0-7.9 | Jar-3 | 6 | 23.5 | | | | | | | |
| | 10.4-10.5 | Jar-4 | 1 | 27.8 | | | | | | | |
| | 12.9-13.0 | Jar-5 | 1 | 27.9 | | | | | | | |
| | 15.4-15.5 | Jar-6 | 1 | 29.1 | | | | | | | |

* Sack samples or disturbed wax samples

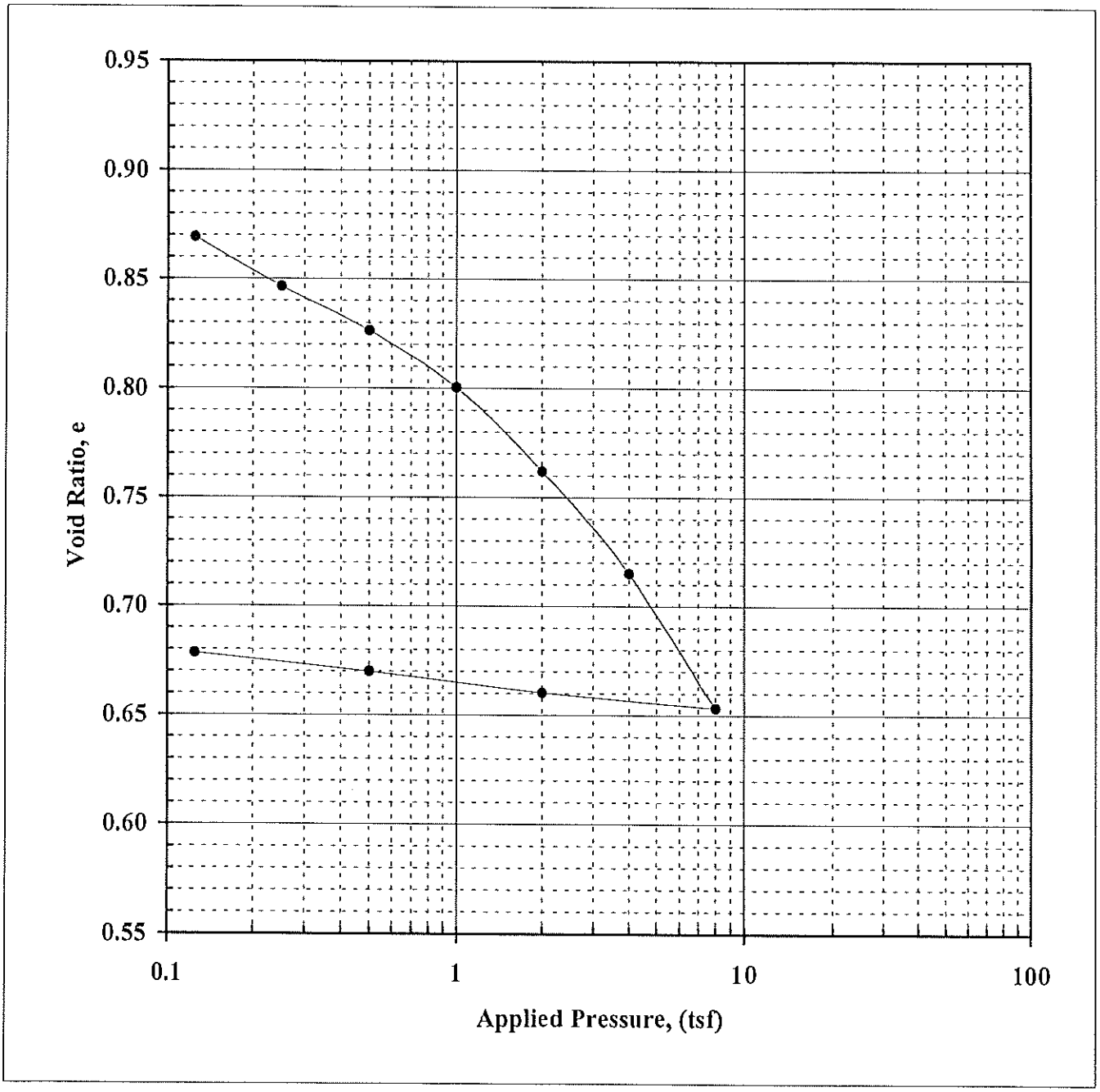
| BORING NO. | SAMPLE DEPTH (feet) | SAMPLE NUMBER | LABORATORY TESTS | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Classification Group Number | Moisture Content, % | Atterberg Limit | | | $\gamma_t$ (pcf) | Unconfined Compression | Standard Proctor | |
| | | | | | Liquid Limit | Plastic Limit | Plasticity Index | | qu, (tsf) | $\gamma_{dmax}$ (pcf) | Optimum Moisture Content, % |
| AD-4 | 0.5-1.0 | Jar-1 | 5 | 24.6 | | | | | | | |
| | 2.5-2.8 | Jar-2 | 5 | 15.1 | | | | | | | |
| | 4.5-5.7 | Jar-3 | 3 | 25.0 | Not Enough Materials | | | | | | |
| | 6.5-8.0 | Jar-4 | 6 | 26.6 | | | | | | | |
| | 8.5-10.0 | Jar-5 | 1 | 29.0 | | | | | | | |
| | 10.5-12.0 | Jar-6 | 1 | 27.9 | | | | | | | |
| | 12.5-14.0 | Jar-7 | 10 | 28.5 | | | | | | | |
| | 14.5-16.0 | Jar-8 | 10 | 29.9 | | | | | | | |
| ADU-5 | 2.0-3.5 | WAX-1 | | 20.1 | 41 | 19 | 22 | 106.6 | 1.46 | | |
| | 3.5-3.6 | Jar-1 | 5 | 18.6 | | | | | | | |
| | 4.5-6.5 | WAX-2 | | 29.6 | 40 | 20 | 20 | 113.6 | | | |
| | 6.4-6.5 | Jar-2 | 5 | 23.6 | | | | | | | |
| | 7.0-8.9 | WAX-3 | | 20.5 | 40 | 19 | 21 | 102.7 | 0.54 | | |
| | 8.9-9.0 | Jar-3 | 1 | 21.9 | 40 | 20 | 20 | | | | |
| | 9.5-11.4 | WAX-4 | | 23.3 | 39 | 21 | 18 | 100.2 | 0.49 | | |
| | 11.4-11.5 | Jar-4 | 4 | 26.0 | 57 | 22 | 35 | | | | |
| | 13.9-14.0 | Jar-5 | 6 | 32.0 | | | | | | | |
| | 16.4-16.5 | Jar-6 | 6 | 33.2 | | | | | | | |
| | 18.9-19.0 | Jar-7 | 6 | 32.6 | | | | | | | |
| | 21.4-21.5 | Jar-8 | 1 | 29.6 | | | | | | | |
| | 23.9-24.0 | Jar-9 | 10 | 27.1 | | | | | | | |
| | 26.4-26.5 | Jar-10 | 10 | 28.1 | | | | | | | |
| | 28.9-29.0 | Jar-11 | 10 | 29.9 | | | | | | | |
| | 31.4-35.0 | Jar-12 | 10 | 28.3 | | | | | | | |
| | 33.9-34.0 | Jar-13 | 1 | 26.2 | | | | | | | |

| BORING NO. | SAMPLE DEPTH (feet) | SAMPLE NUMBER | LABORATORY TESTS | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Classification Group Number | Moisture Content, % | Atterberg Limit | | | $\gamma_t$ (pcf) | Unconfined Compression | Standard Proctor | |
| | | | | | Liquid Limit | Plastic Limit | Plasticity Index | | qu, (tsf) | $\gamma_{dmax}$ (pcf) | Optimum Moisture Content, % |
| | | | | | | | | | | | |
| | 36.4-36.5 | Jar-14 | 10 | 25.7 | | | | | | | |
| | 38.9-39.0 | Jar-15 | 10 | 25.8 | | | | | | | |
| | 41.4-41.5 | Jar-16 | 10 | 23.4 | | | | | | | |
| | 43.9-44.0 | Jar-17 | 2 | 18.7 | Non-plastic | | | | | | |
| | 46.4-46.5 | Jar-18 | 2 | 17.0 | | | | | | | |
| | 49.0-49.5 | Jar-19 | 3 | 18.7 | | | | | | | |
| | 51.5-52.4 | Jar-20 | 2 | 20.3 | | | | | | | |
| | 54.0-54.6 | Jar-21 | 2 | 13.7 | | | | | | | |
| | 54.6-55.5 | Jar-22 | 4 | 21.7 | | | | | | | |
| ADU-6 | 0.5-2.4 | WAX-1 | | 19.4 | 40 | 20 | 20 | 104.2 | 0.97 | | |
| | 2.4-2.5 | Jar-1 | 6 | 23.8 | | | | | | | |
| | 4.9-5.0 | Jar-2 | 5 | 20.6 | | | | | | | |
| | 5.5-7.4 | WAX-3 | | 23.0 | 39 | 20 | 19 | N/T* | N/T* | | |
| | 7.4-7.5 | Jar-3 | 10 | 23.0 | | | | | | | |
| | 9.9-10.0 | Jar-4 | 8 | 21.2 | 30 | 20 | 10 | | | | |
| | 12.4-12.5 | Jar-5 | 10 | 22.9 | | | | | | | |
| | 13.0-14.9 | WAX-6 | | 23.2 | 29 | 21 | 8 | 100.5 | 0.44 | | |
| | 14.9-15.0 | Jar-6 | 10 | 21.2 | | | | | | | |
| | 17.4-17.5 | Jar-7 | 10 | 22.5 | 37 | 21 | 16 | | | | |
| | 19.9-20.0 | Jar-8 | 10 | 20.1 | | | | | | | |
| | 22.4-22.5 | Jar-9 | 10 | 21.5 | | | | | | | |
| | 24.9-25.0 | Jar-10 | 10 | 22.3 | | | | | | | |
| | 25.5-25.8 | Jar-11 | 7 | 76.2 | Not Enough Materials | | | | | | |

| | | | | | |
|---|---|---|---|---|---|
| Boring No. | ADU-5 | GEOTECHNOLOGY | | Job No. | 06808.01.32KC |
| Sample No. | Wax-2 | ENGINEERING AND ENVIRONMENTAL SERVICES | | Job Name | Blacksnake Creek |
| Depth (ft) | 4.5-6.5' | SAINT LOUIS, COLLINSVILLE, KANSAS CITY | | Sheet No. | |
| Tested By | SR | **One-Dimensional Consolidation Test** | | Test Date | 07/19/04 |
| Computed By | yaw | (ASTM D 4546-96-C) | | Checked By | ARK |

**Plot of Void Ratio versus Log of Pressure**

| | | | | |
|---|---|---|---|---|
| **Compression Index, $C_c$ =** | 0.138 | **Void Ratio, $e_o$ =** | 0.9222 | |
| **Recompression Index, $C_r$ =** | 0.013 | **Preconsolidation Pressure =** | 1.6 | (tsf) |



| | | | |
|---|---|---|---|
| Boring No. | ADU-5 | b No. | 06808.01.32KC |
| Sample No. | Wax-2 | b Name | Blacksnake Creek |
| Depth (ft) | 4.5-6.5' | ıeet No. | |
| Tested By | SR | Test Date | 07/19/04 |
| Computed By | yaw | Checked B | ARK |

**GEOTECHNOLOGY**
ENGINEERING AND ENVIRONMENTAL SERVICES
SAINT LOUIS, COLLINSVILLE, KANSAS CITY

## One-Dimensional Consolidation Test

(ASTM D 4546-96-C)

Summary and Computation Sheet

SAMPLE DESCRIPTION: CL, brown, low plasticity CLAY

Assumed Specific Gravity Gs: 2.70

| Ring No. #1 | BEFORE TEST | AFTER TEST |
|---|---|---|
| Height | 1.0000 (in.) | 0.8732 (in.) |
| Diameter | 2.5000 (in.) | 2.5000 (in.) |
| Area | 4.9087 (sq.in.) | 4.9087 (sq.in.) |
| Volume | 4.9087 (cu.in.) | 4.2863 (cu.in.) |
| Wt. (ring + wet soil) | 362.84 (gm) | 360.12 (gm) |
| Wt. (ring + dry soil) | 329.41 (gm) | 329.41 (gm) |
| Wt. of ring | 216.47 (gm) | 216.47 (gm) |
| Wt. of wet soil | 146.37 (gm) | 143.65 (gm) |
| Wt. of dry soil | 112.94 (gm) | 112.94 (gm) |
| Moisture content | 29.6 (%) | 27.2 (%) |
| Wet density | 113.6 (pcf) | 127.7 (pcf) |
| Dry density | 87.7 (pcf) | 100.4 (pcf) |
| Solids height | 0.5203 (in.) | 0.5202 (in.) |
| Ht. of water | 0.4157 (in.) | 0.3819 (in.) |
| Saturation | 82 (%) | 100 (%) |

Dial Reading Factor: 0.0001 (in.)
Initial Dial Reading: 13
Method for Final Reading (1. Use $D_{100}$, 2. D at end of load, 3. D at specific time): 2

| Elaspsed Time (min) | Pressure Increment (tsf) | Dial Reading | Correc-tion | Dial Reading Corrected | Change of Reading | Specimen Height (in.) | Void Ratio | Average Pressure (tsf) | Cv (in.$^2$/min) |
|---|---|---|---|---|---|---|---|---|---|
| | 0 | 13 | 0 | 13 | 0 | 1.0000 | 0.922 | | |
| | 0.125 | 295 | 7 | 288 | 275 | 0.9725 | 0.869 | | |
| | 0.250 | 427 | 21 | 406 | 118 | 0.9607 | 0.847 | | |
| | 0.500 | 548 | 37 | 511 | 105 | 0.9502 | 0.826 | | |
| | 1.000 | 700 | 54 | 646 | 135 | 0.9367 | 0.800 | | |
| | 2.000 | 922 | 76 | 846 | 200 | 0.9167 | 0.762 | | |
| | 4.000 | 1192 | 102 | 1090 | 244 | 0.8923 | 0.715 | | |
| | 8.000 | 1531 | 119 | 1412 | 322 | 0.8601 | 0.653 | | |
| | 2.000 | 1479 | 104 | 1375 | -37 | 0.8638 | 0.660 | | |
| | 0.500 | 1414 | 89 | 1325 | -50 | 0.8688 | 0.670 | | |
| | 0.125 | 1348 | 67 | 1281 | -44 | 0.8732 | 0.678 | | |

| | | | |
|---|---|---|---|
| Boring No. | ADU-5 | Job No. | 06808.01.32KC |
| Sample No. | Wax-2 | Job Name | Blacksnake Creek |
| Depth (ft) | 4.5-6.5' | | |
| Tested By | SR | Test Date | 07/19/04 |
| Computed By | yaw | Checked By | AKU |

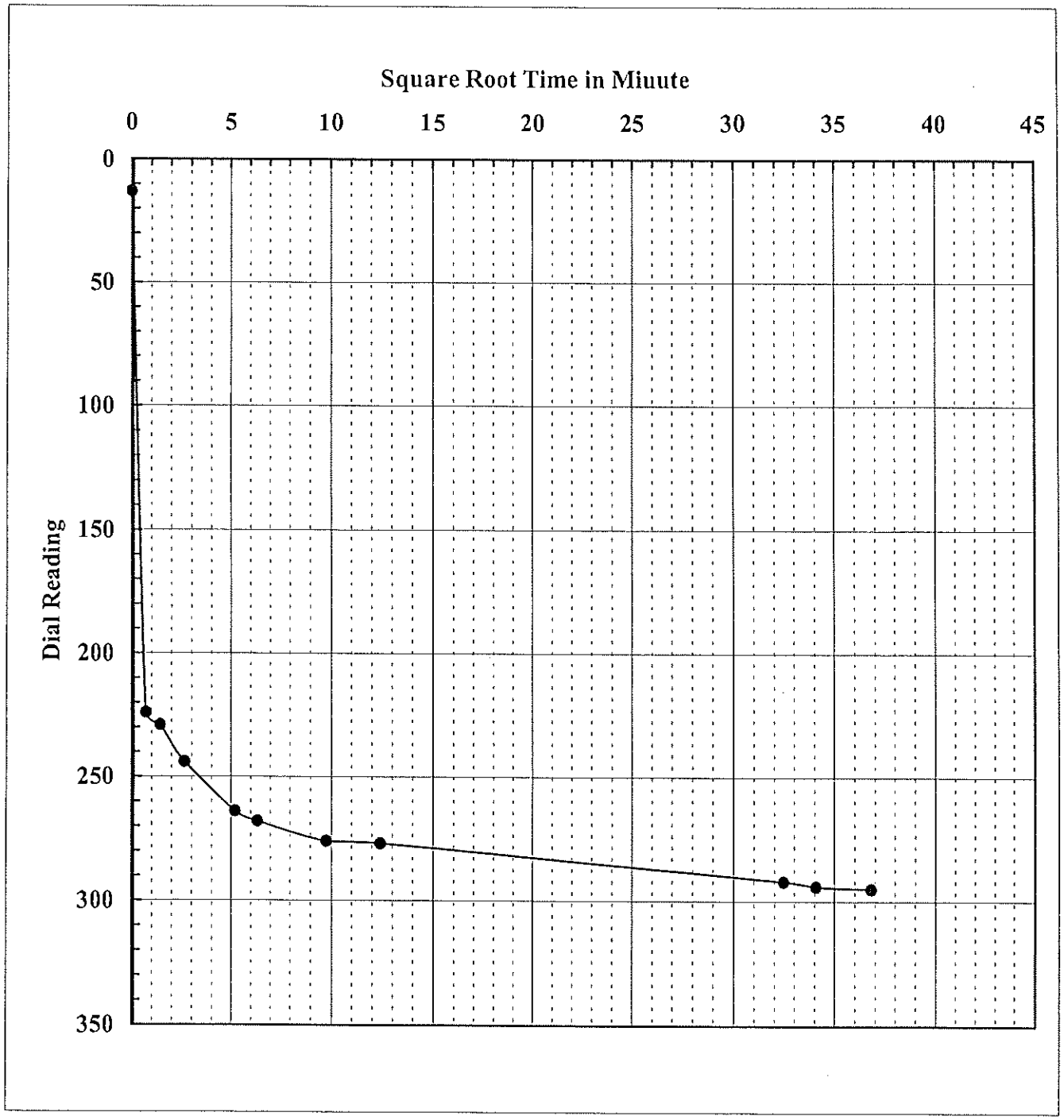
GEOTECHNOLOGY
ENGINEERING AND ENVIRONMENTAL SERVICES
SAINT LOUIS, COLLINSVILLE, KANSAS CITY

## One-Dimensional Consolidation Test

(ASTM D 4546-96-C)

Plot of Deformation versus Square Root of Time

Applied Pressure = 0.125 (tsf)



| | | | |
|---|---|---|---|
| Boring No. | ADU-5 | Job No. | 06808.01.32KC |
| Sample No. | Wax-2 | Job Name | Blacksnake Creek |
| Depth (ft) | 4.5-6.5' | | |
| Tested By | SR | Test Date | 07/19/04 |
| Computed By | yaw | Checked By | AKC |

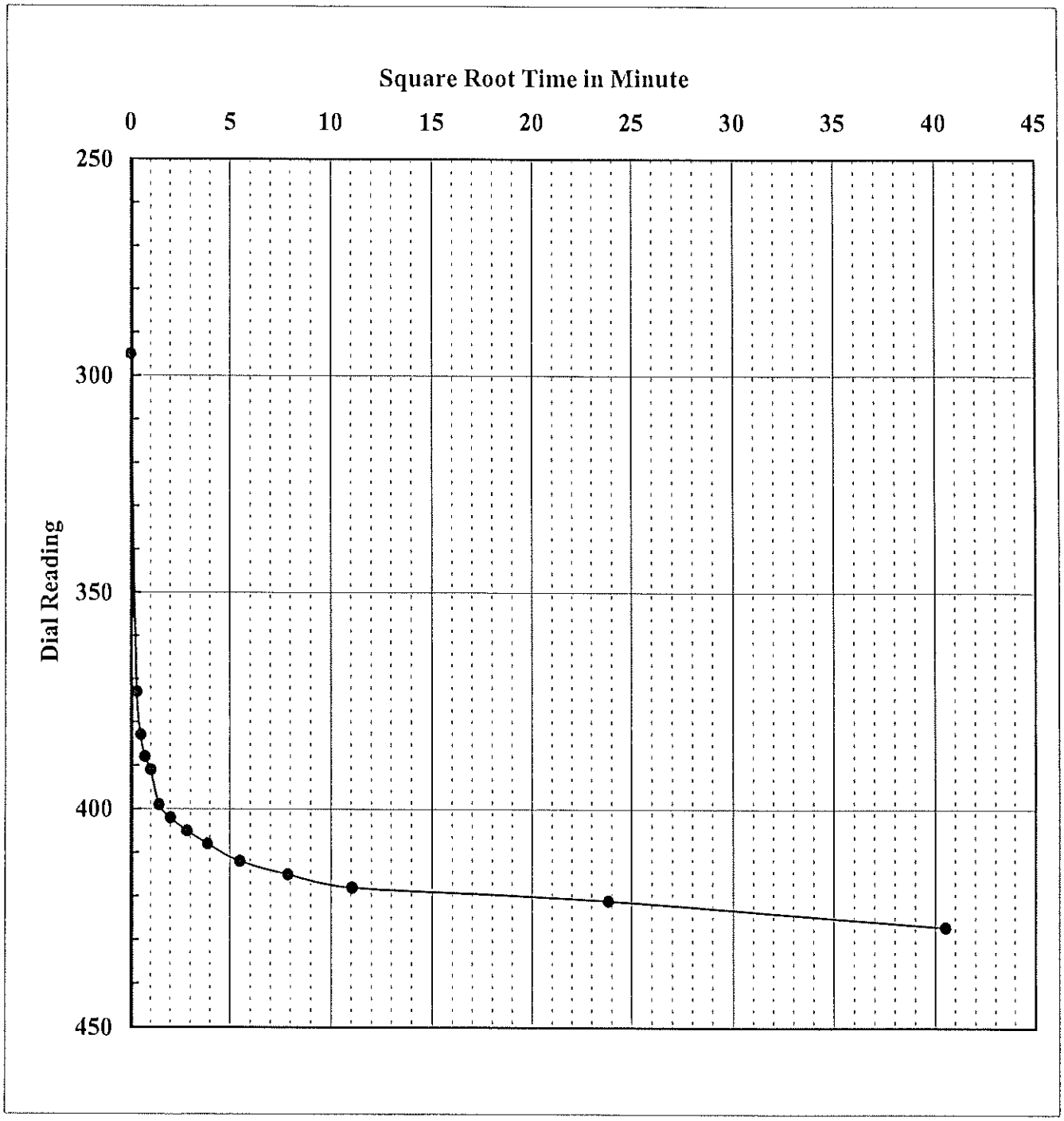
**GEOTECHNOLOGY**
ENGINEERING AND ENVIRONMENTAL SERVICES
SAINT LOUIS, COLLINSVILLE, KANSAS CITY

## One-Dimensional Consolidation Test

(ASTM D 4546-96-C)

**Plot of Deformation versus Square Root of Time**

**Applied Pressure = 0.25 (tsf)**



| | | | |
|---|---|---|---|
| Boring No. | ADU-5 | Job No. | 06808.01.32KC |
| Sample No. | Wax-2 | Job Name | Blacksnake Creek |
| Depth (ft) | 4.5-6.5' | Sheet No. | |
| Tested By | SR | Test Date | 07/19/04 |
| Computed By | yaw | Checked By | ARK |

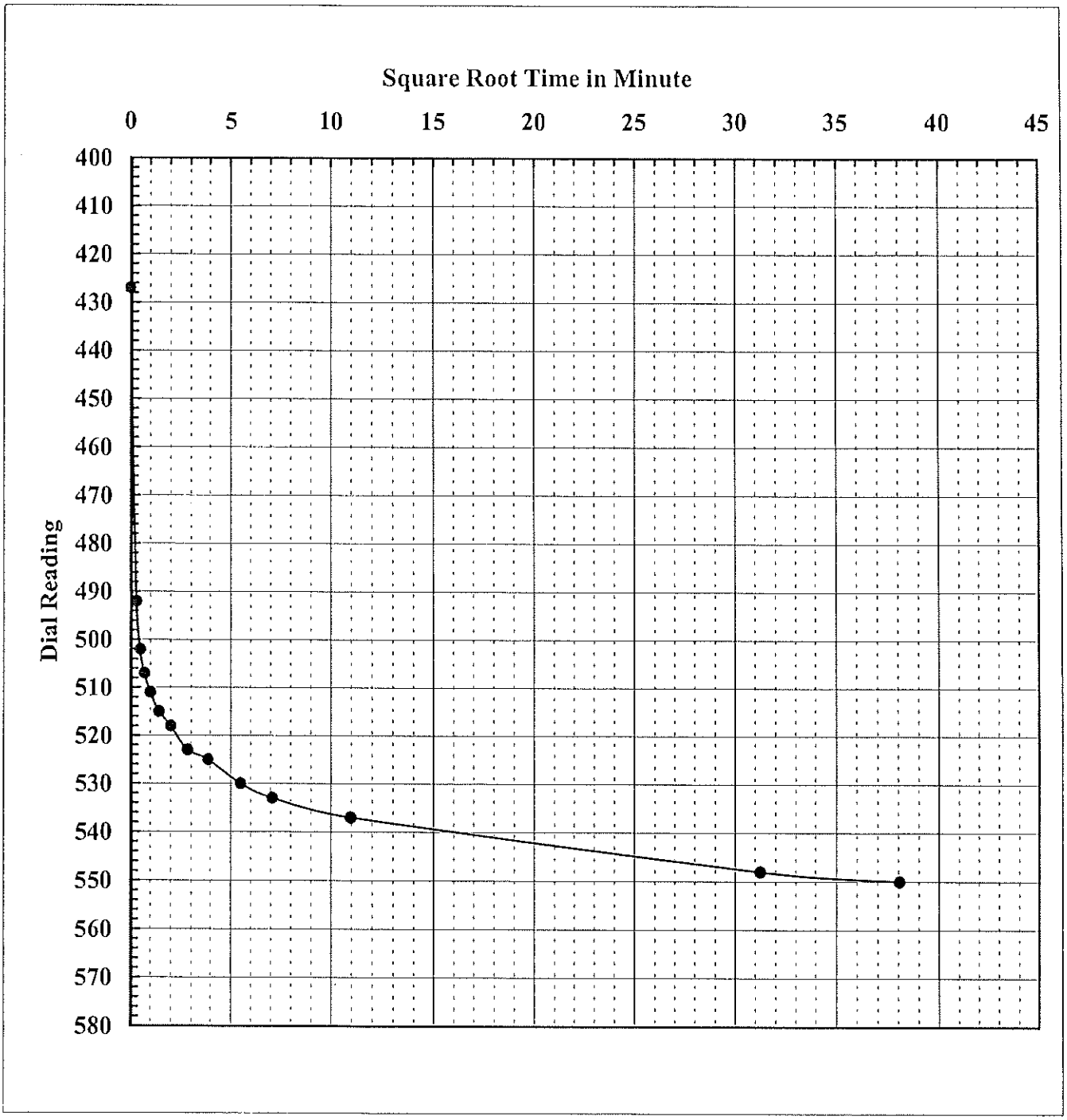
GEOTECHNOLOGY
ENGINEERING AND ENVIRONMENTAL SERVICES
SAINT LOUIS, COLLINSVILLE, KANSAS CITY

## One-Dimensional Consolidation Test

(ASTM D 4546-96-C)

Plot of Deformation versus Square Root of Time

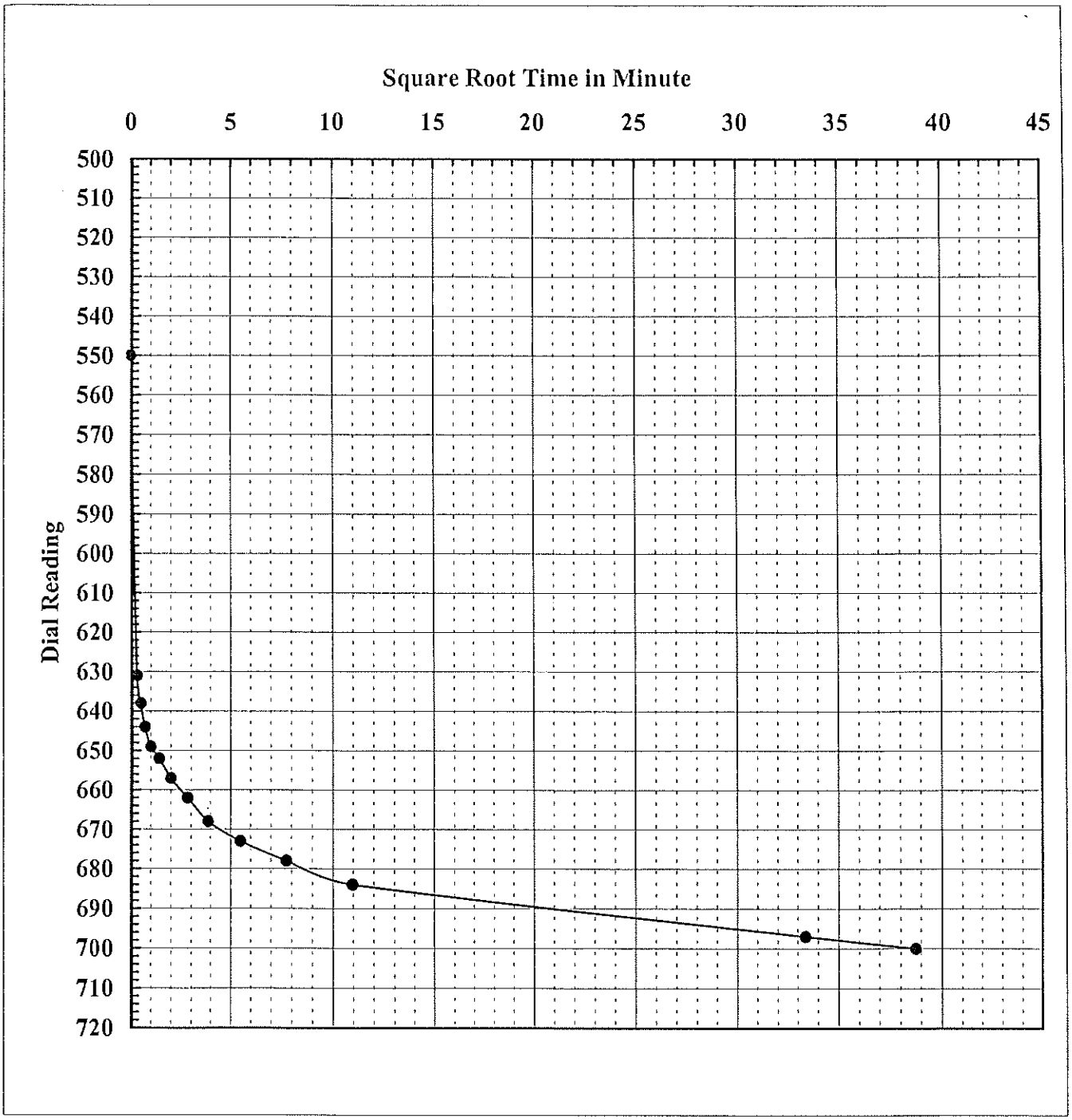
Applied Pressure = 0.5 (tsf)



| | | | | | |
|---|---|---|---|---|---|
| Boring No. | ADU-5 | GEOTECHNOLOGY | | Job No. | 06808.01.32KC |
| Sample No. | Wax-2 | ENGINEERING AND ENVIRONMENTAL SERVICES | | Job Name | Blacksnake Creek |
| Depth (ft) | 4.5-6.5' | SAINT LOUIS, COLLINSVILLE, KANSAS CITY | | Sheet No. | |
| Tested By | SR | **One-Dimensional Consolidation Test** | | Test Date | 07/19/04 |
| Computed By | yaw | (ASTM D 4546-96-C) | | Checked By | |

**Plot of Deformation versus Square Root of Time**

**Applied Pressure = 1 (tsf)**



| | | | |
|---|---|---|---|
| Boring No. | ADU-5 | Job No. | 06808.01.32KC |
| Sample No. | Wax-2 | Job Name | Blacksnake Creek |
| Depth (ft) | 4.5-6.5' | Sheet No. | |
| Tested By | SR | Test Date | 07/19/04 |
| Computed By | yaw | Checked By | |

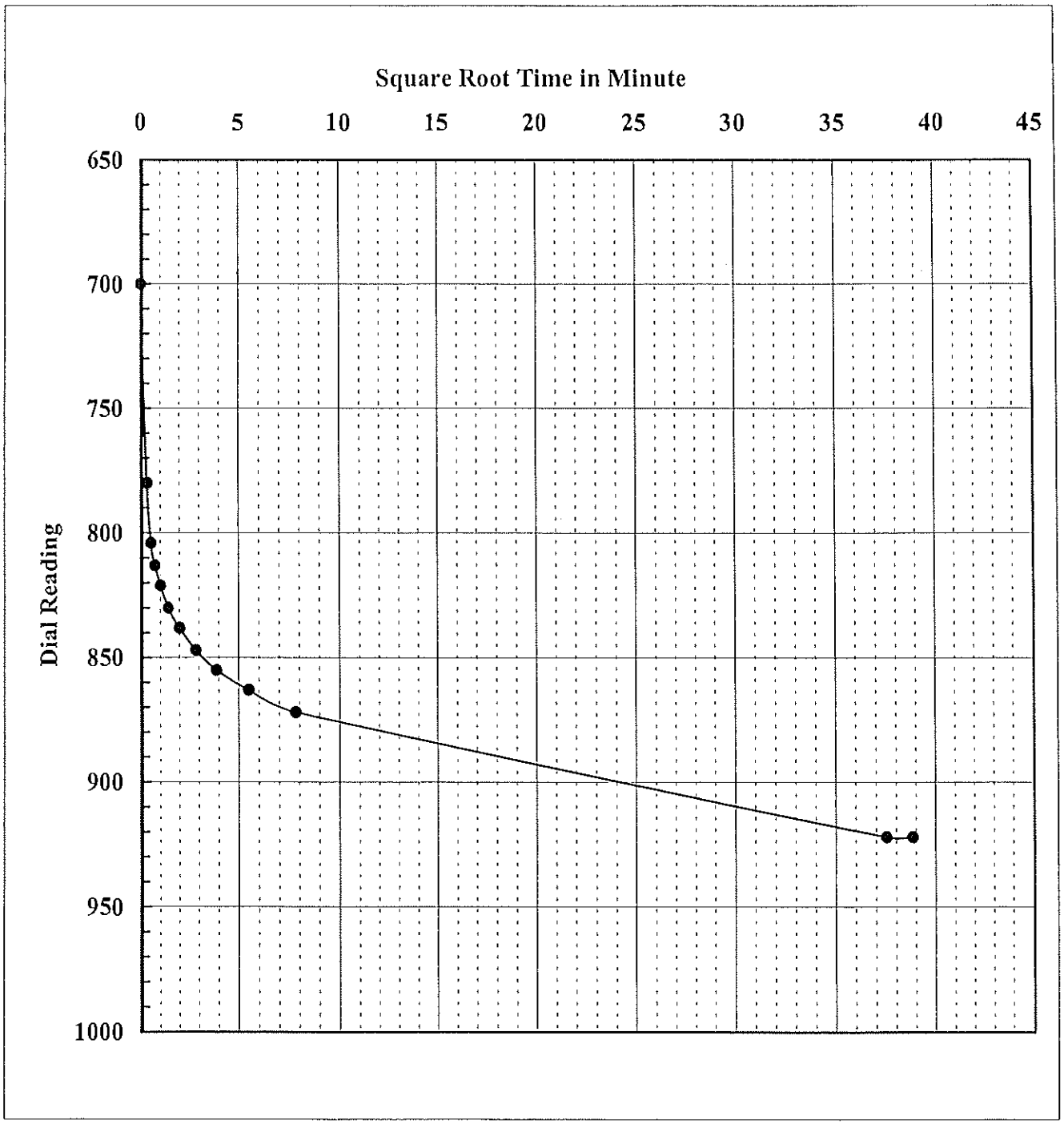
GEOTECHNOLOGY
ENGINEERING AND ENVIRONMENTAL SERVICES
SAINT LOUIS, COLLINSVILLE, KANSAS CITY

## One-Dimensional Consolidation Test

(ASTM D 4546-96-C)

Plot of Deformation versus Square Root of Time

Applied Pressure = 2 (tsf)



| | | | |
|---|---|---|---|
| Boring No. | ADU-5 | Job No. | 06808.01.32KC |
| Sample No. | Wax-2 | Job Name | Blacksnake Creek |
| Depth (ft) | 4.5-6.5' | Sheet No. | |
| Tested By | SR | Test Date | 07/19/04 |
| Computed By | yaw | Checked By | Arcu |

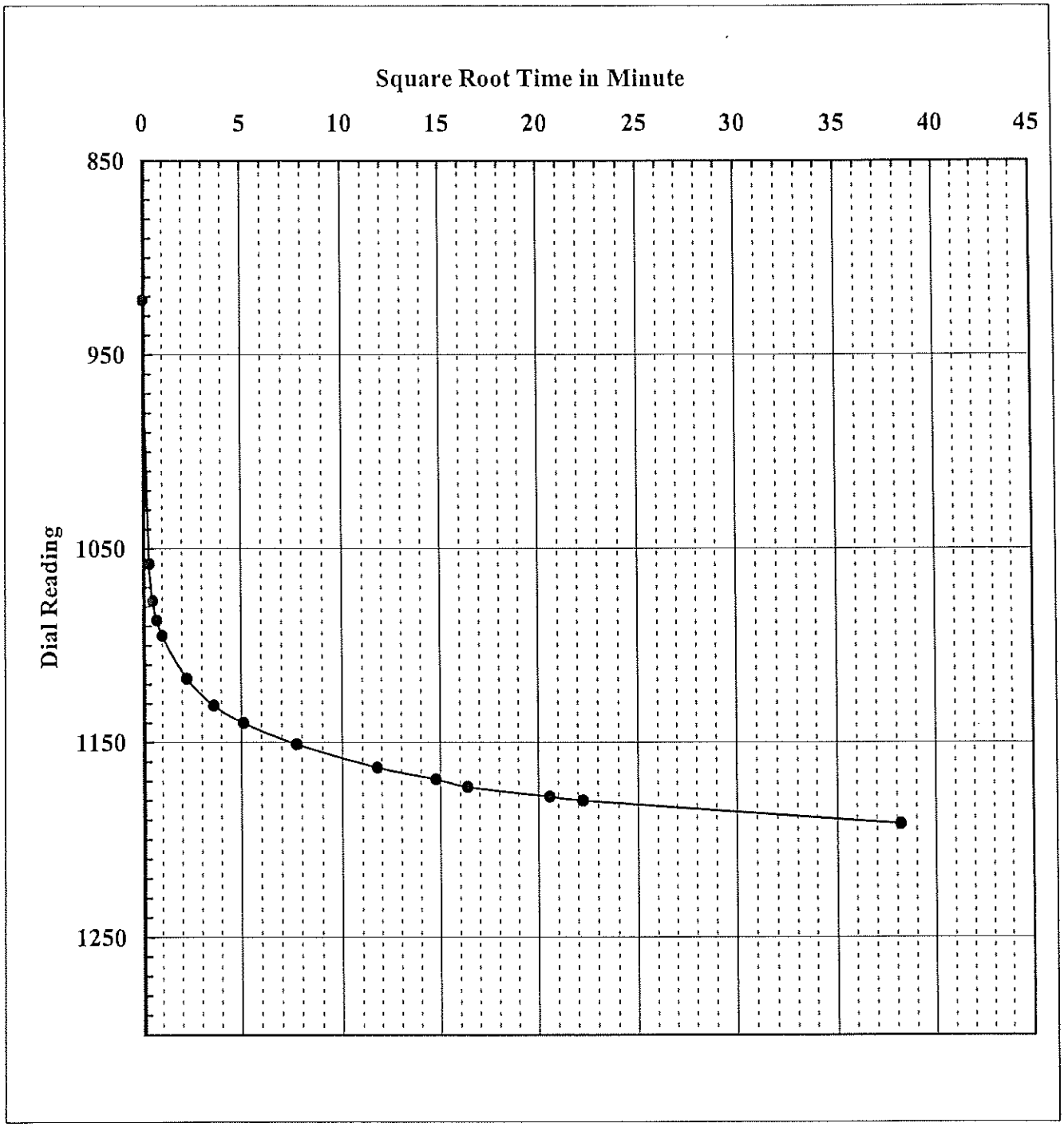
GEOTECHNOLOGY
ENGINEERING AND ENVIRONMENTAL SERVICES
SAINT LOUIS, COLLINSVILLE, KANSAS CITY

## One-Dimensional Consolidation Test

(ASTM D 4546-96-C)

**Plot of Deformation versus Square Root of Time**

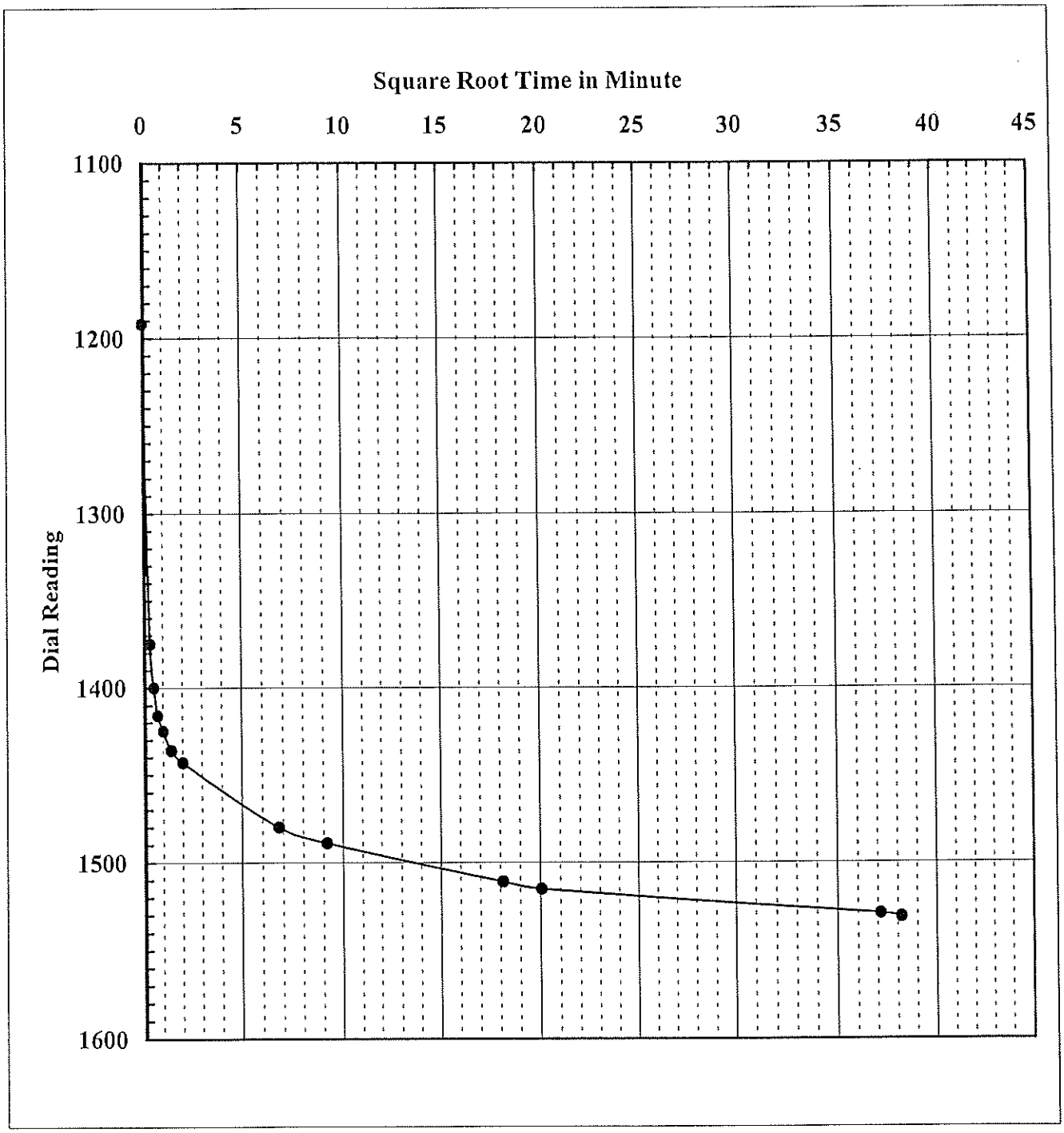
**Applied Pressure = 4 (tsf)**



| | | | | | |
|---|---|---|---|---|---|
| Boring No. | ADU-5 | GEOTECHNOLOGY | | Job No. | 06808.01.32KC |
| Sample No. | Wax-2 | ENGINEERING AND ENVIRONMENTAL SERVICES | | Job Name | Blacksnake Creek |
| Depth (ft) | 4.5-6.5' | SAINT LOUIS, COLLINSVILLE, KANSAS CITY | | Sheet No. | |
| Tested By | SR | **One-Dimensional Consolidation Test** | | Test Date | 07/19/04 |
| Computed By | yaw | (ASTM D 4546-96-C) | | Checked By | AKK |

**Plot of Deformation versus Square Root of Time**

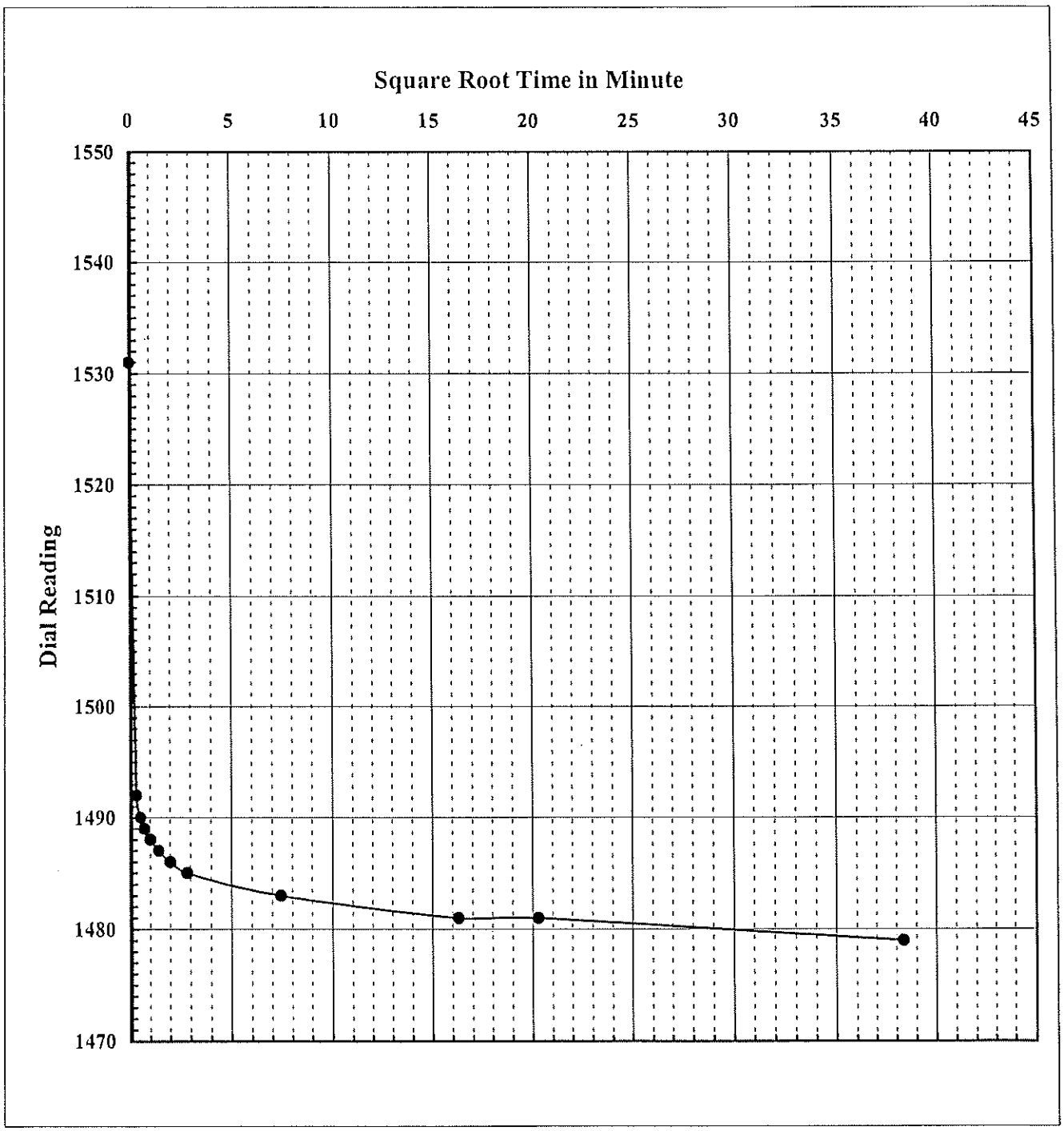
**Applied Pressure = 8 (tsf)**



| | | | | | |
|---|---|---|---|---|---|
| Boring No. | ADU-5 | GEOTECHNOLOGY | | Job No. | 06808.01.32KC |
| Sample No. | Wax-2 | ENGINEERING AND ENVIRONMENTAL SERVICES | | Job Name | Blacksnake Creek |
| Depth (ft) | 4.5-6.5' | SAINT LOUIS, COLLINSVILLE, KANSAS CITY | | Sheet No. | |
| Tested By | SR | **One-Dimensional Consolidation Test** | | Test Date | 07/19/04 |
| Computed By | yaw | (ASTM D 4546-96-C) | | Checked By | AKK |

**Plot of Deformation versus Square Root of Time**

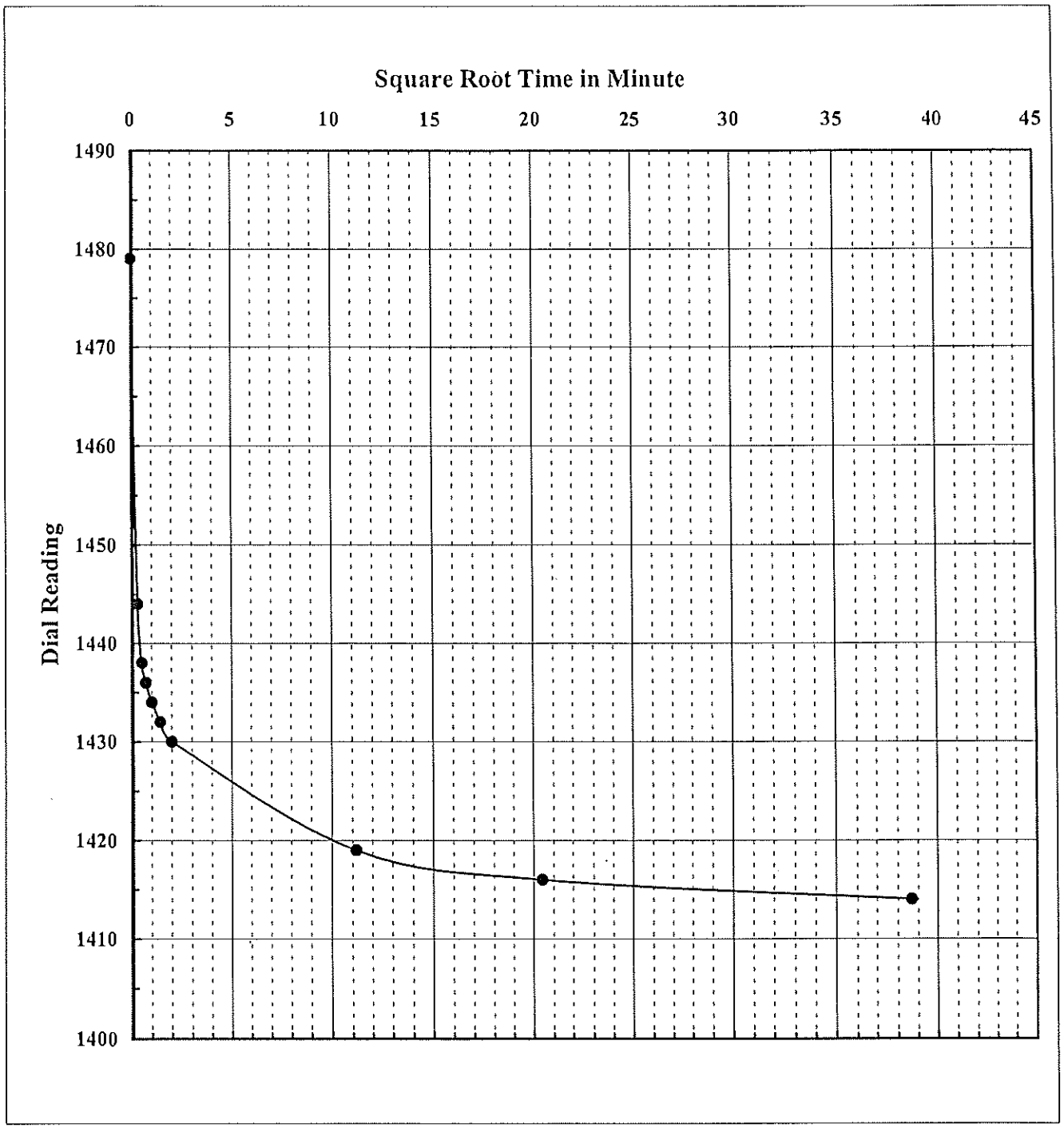
**Applied Pressure @ Unloading = 2 (tsf)**



| | | | | | |
|---|---|---|---|---|---|
| Boring No. | ADU-5 | GEOTECHNOLOGY | | Job No. | 06808.01.32KC |
| Sample No. | Wax-2 | ENGINEERING AND ENVIRONMENTAL SERVICES | | Job Name | Blacksnake Creek |
| Depth (ft) | 4.5-6.5' | SAINT LOUIS, COLLINSVILLE, KANSAS CITY | | Sheet No. | |
| Tested By | SR | **One-Dimensional Consolidation Test** | | Test Date | 07/19/04 |
| Computed By | yaw | (ASTM D 4546-96-C) | | Checked By | ARK |

**Plot of Deformation versus Square Root of Time**

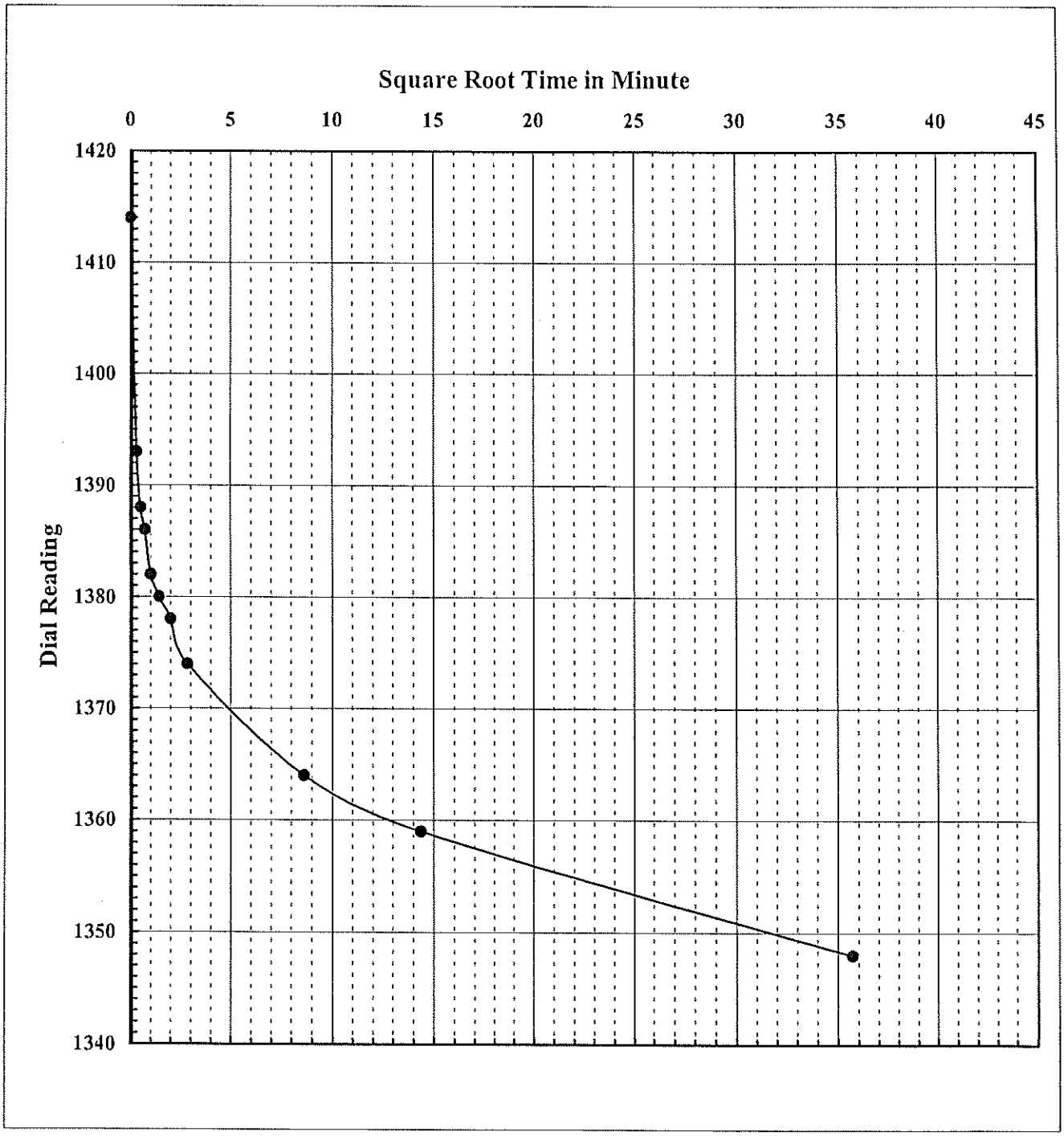
**Applied Pressure @ Unloading = 0.5 (tsf)**



| | | | | | |
|---|---|---|---|---|---|
| Boring No. | ADU-5 | GEOTECHNOLOGY | | Job No. | 06808.01.32KC |
| Sample No. | Wax-2 | ENGINEERING AND ENVIRONMENTAL SERVICES | | Job Name | Blacksnake Creek |
| Depth (ft) | 4.5-6.5' | SAINT LOUIS, COLLINSVILLE, KANSAS CITY | | Sheet No. | |
| Tested By | SR | **One-Dimensional Consolidation Test** | | Test Date | 07/19/04 |
| Computed By | yaw | (ASTM D 4546-96-C) | | Checked By | AJK |

**Plot of Deformation versus Square Root of Time**

**Applied Pressure @ Unloading = 0.125 (tsf)**



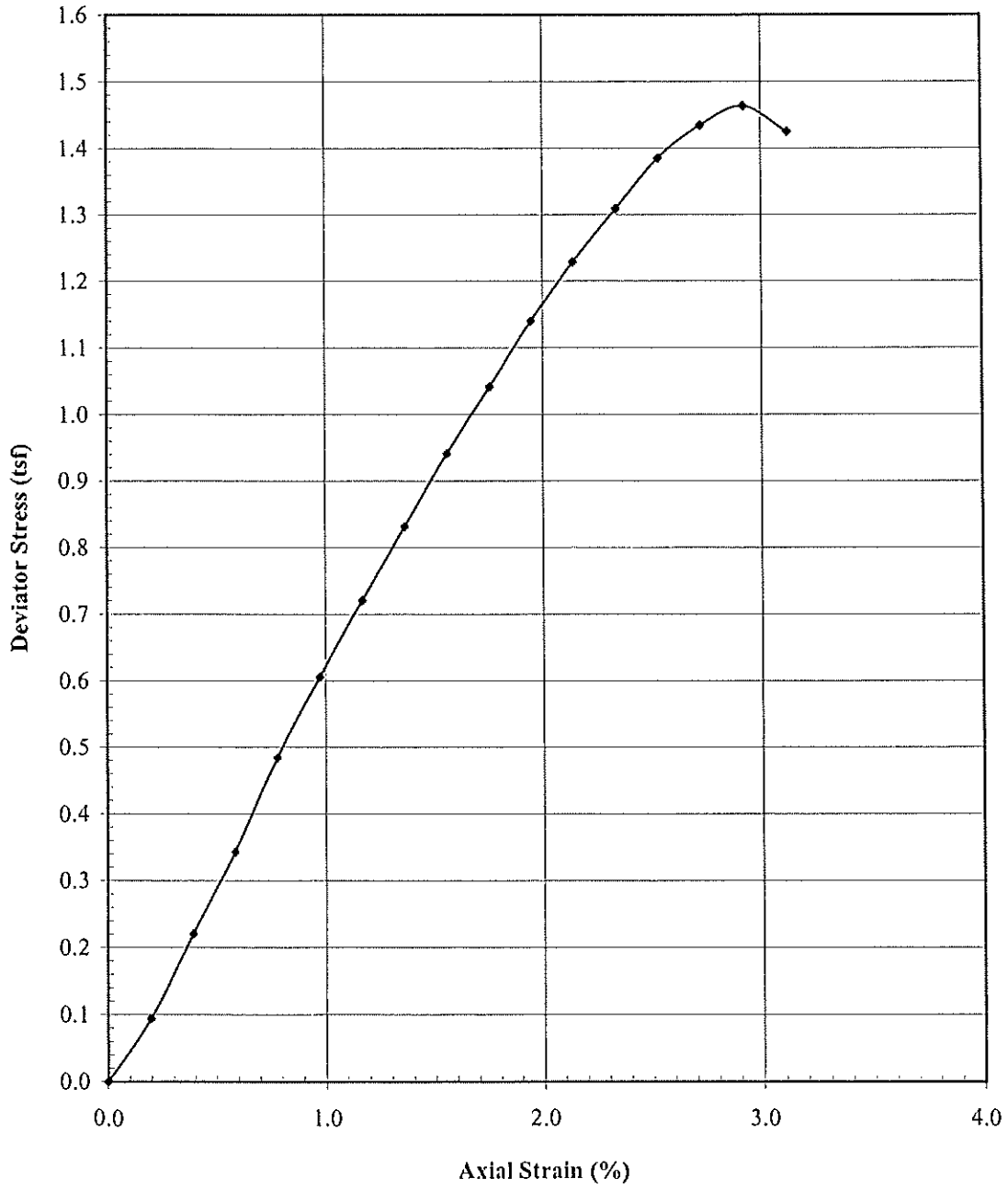
| | |
|---|---|
| Boring No. | ADU-5 |
| Sample No. | Wax-1 |
| Depth (ft) | 2-3.5' |
| Date | 07/09/2004 |
| Sheet No. | |

**GEOTECHNOLOGY**
ENGINEERING AND ENVIRONMENTAL SERVICES
SAINT LOUIS, COLLINSVILLE, KANSAS CITY

**UNCONFINED COMPRESSION TEST**
**ASTM D-2166**
**Stress-Strain Plot**

| | |
|---|---|
| Job No. | 0680801.32KC |
| Job Name | Black Snake Creek |
| Tested By | JIU |
| Calculated By | yaw |
| Checked By | |

| | | | |
|---|---|---|---|
| Max. Shear Strength = | 1.46 tsf | Water Content = | 20.1 % |
| Failure Strain = | 2.92 % | Dry Unit Weight = | 106.6 pcf |



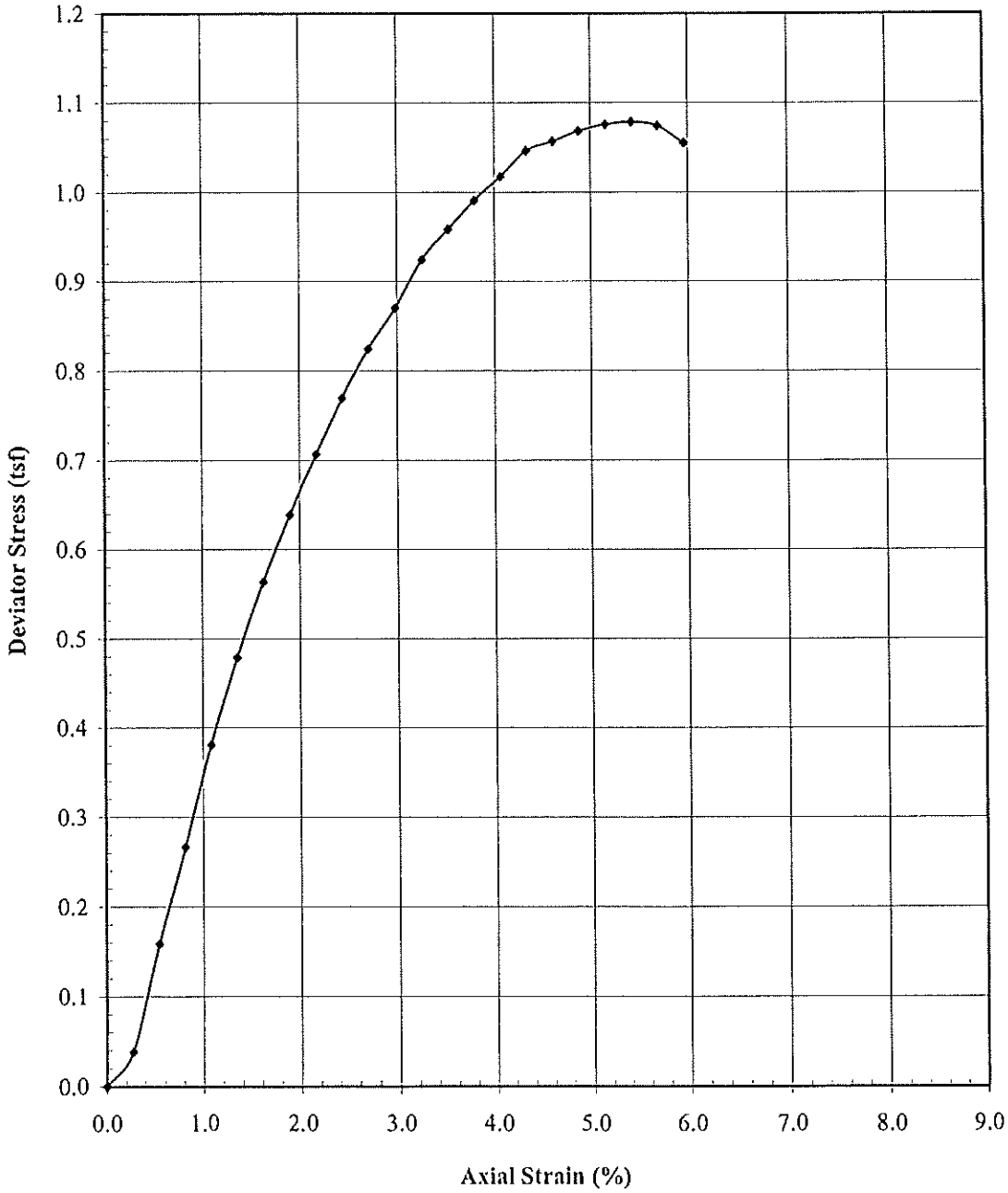
| | |
|---|---|
| Boring No. | ADU-5 |
| Sample No. | Wax-3 |
| Depth (ft) | 7-8.9' |
| Date | 07/09/2004 |
| Sheet No. | |

**GEOTECHNOLOGY**
ENGINEERING AND ENVIRONMENTAL SERVICES
SAINT LOUIS, COLLINSVILLE, KANSAS CITY

**UNCONFINED COMPRESSION TEST**
**ASTM D-2166**
**Stress-Strain Plot**

| | |
|---|---|
| Job No. | 0680801.32KC |
| Job Name | Black Snake Creek |
| Tested By | JIU |
| Calculated By | yaw |
| Checked By | |

| | | | |
|---|---|---|---|
| Max. Shear Strength = | 1.08 tsf | Water Content = | 20.5 % |
| Failure Strain = | 5.42 % | Dry Unit Weight = | 102.7 pcf |



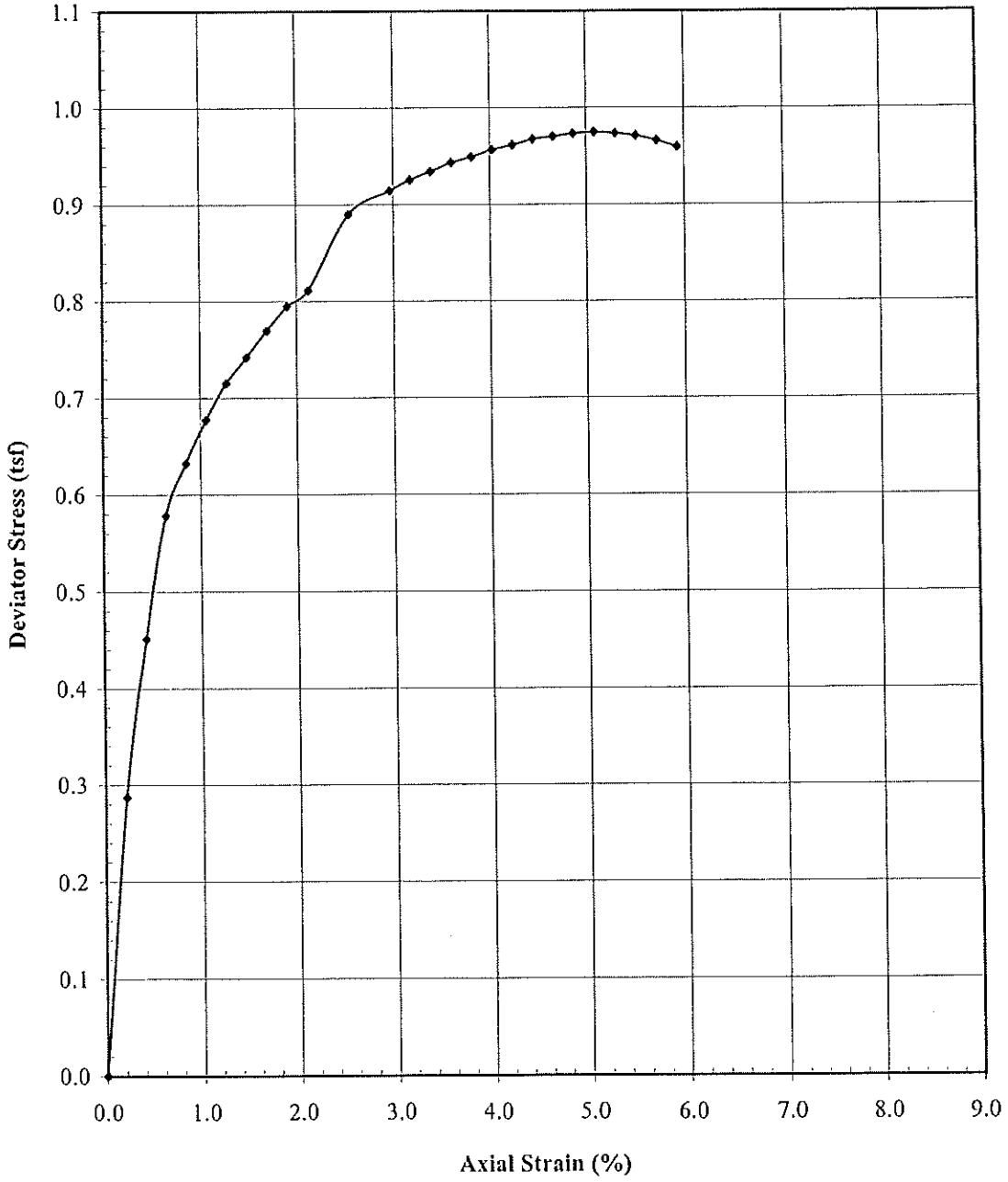
| | |
|---|---|
| Boring No. | ADU-5 |
| Sample No. | Wax-4 |
| Depth (ft) | 9.5-11.4' |
| Date | 06/25/2004 |
| Sheet No. | |

**GEOTECHNOLOGY**
ENGINEERING AND ENVIRONMENTAL SERVICES
SAINT LOUIS, COLLINSVILLE, KANSAS CITY

**UNCONFINED COMPRESSION TEST**
**ASTM D-2166**
**Stress-Strain Plot**

| | |
|---|---|
| Job No. | 0680801.32KC |
| Job Name | Black Snake Creek |
| Tested By | JIU |
| Calculated By | yaw |
| Checked By | |

| | | | |
|---|---|---|---|
| Max. Shear Strength = | 0.97 tsf | Water Content = | 23.3 % |
| Failure Strain = | 5.08 % | Dry Unit Weight = | 100.2 pcf |



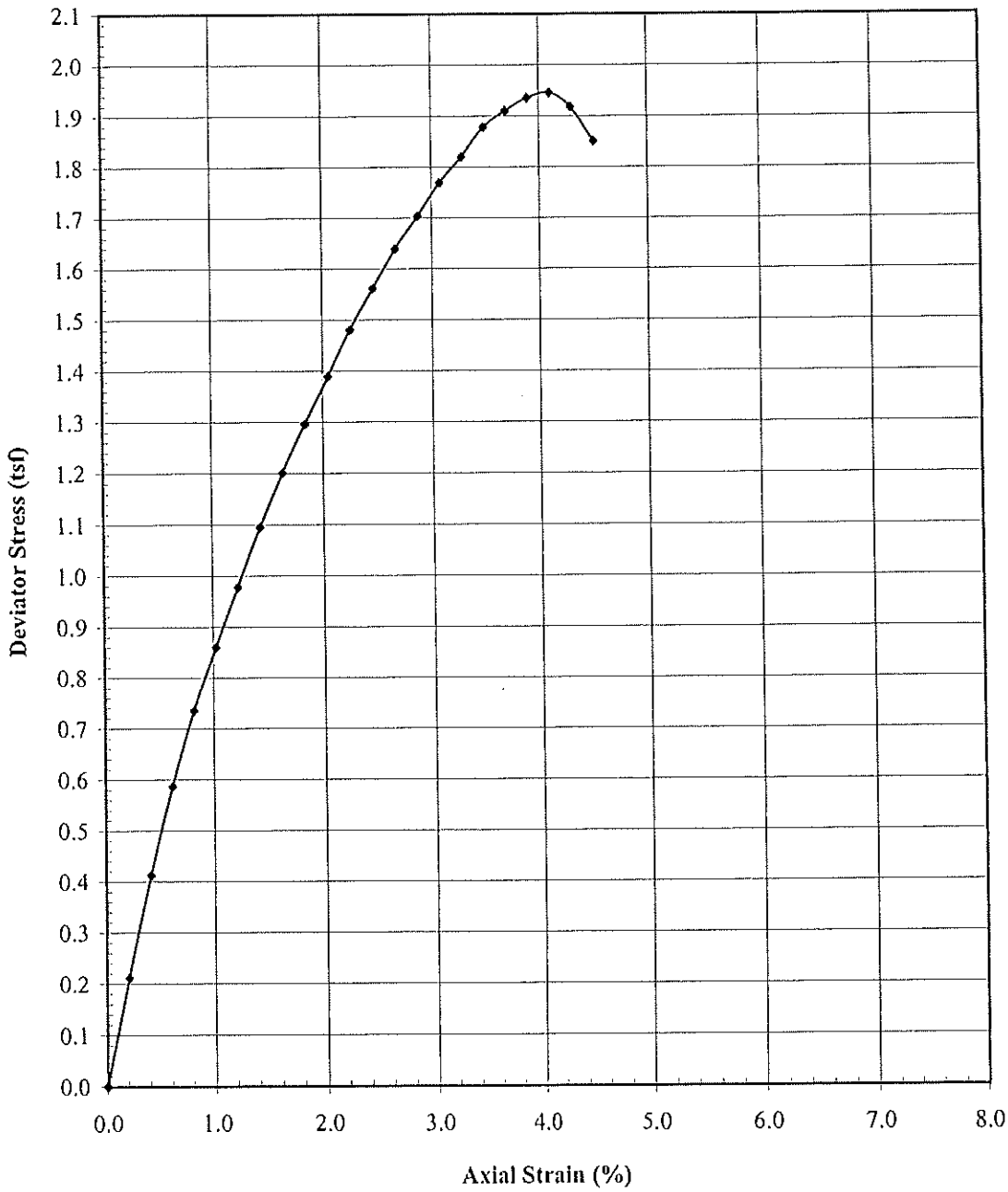
| | |
|---|---|
| Boring No. | ADU-6 |
| Sample No. | Wax-1 |
| Depth (ft) | 0.5-2.4' |
| Date | 06/25/2004 |
| Sheet No. | |

**GEOTECHNOLOGY**
ENGINEERING AND ENVIRONMENTAL SERVICES
SAINT LOUIS, COLLINSVILLE, KANSAS CITY

**UNCONFINED COMPRESSION TEST**
**ASTM D-2166**
**Stress-Strain Plot**

| | |
|---|---|
| Job No. | 0680801.32KC |
| Job Name | Black Snake Creek |
| Tested By | JIU |
| Calculated By | yaw |
| Checked By | GM |

| | | | |
|---|---|---|---|
| Max. Shear Strength = | 1.95 tsf | Water Content = | 19.4 % |
| Failure Strain = | 4.10 % | Dry Unit Weight = | 104.2 pcf |



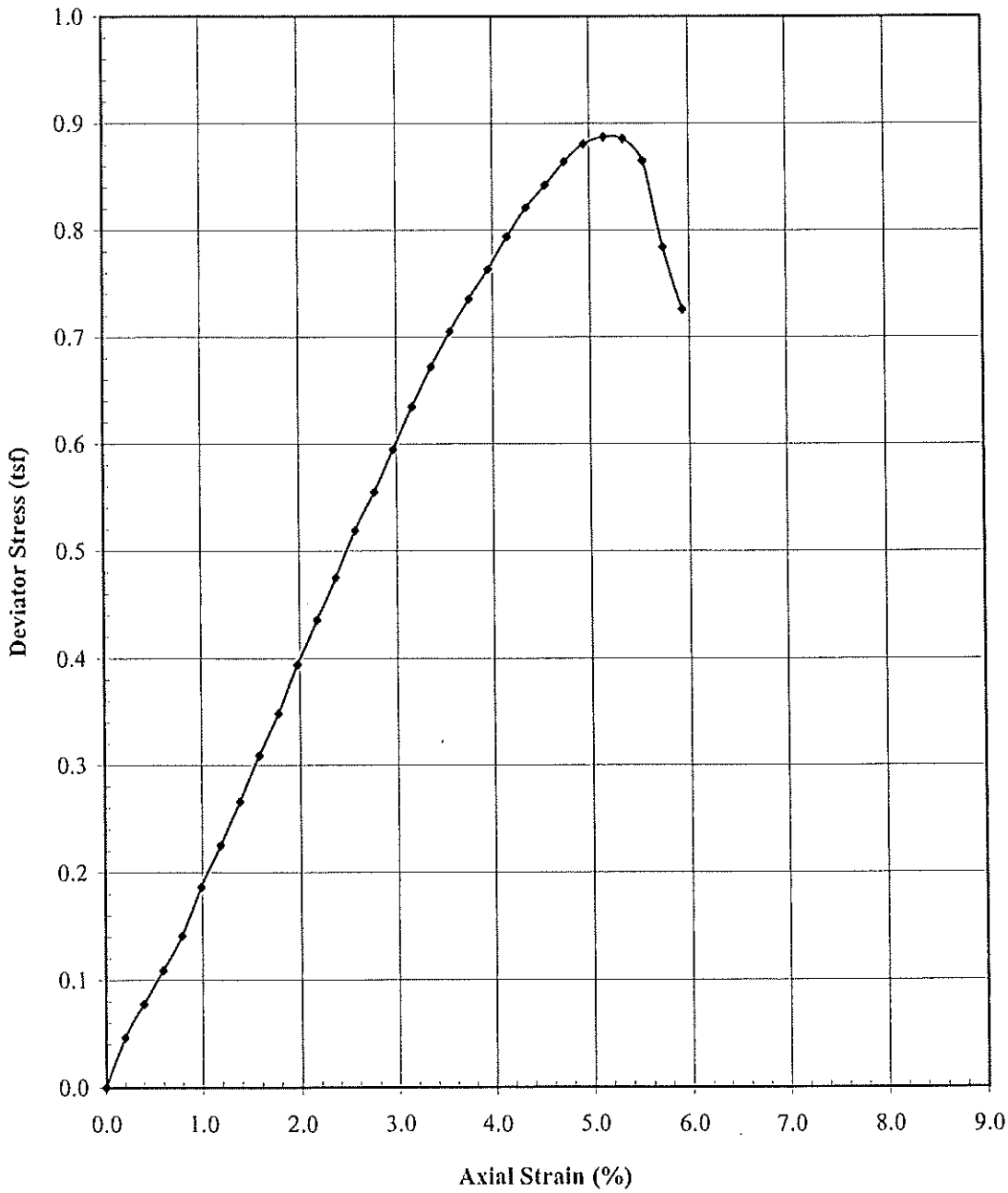
| | |
|---|---|
| Boring No. | ADU-6 |
| Sample No. | Wax-6 |
| Depth (ft) | 13.0-14.9' |
| Date | 06/25/2004 |
| Sheet No. | |

**GEOTECHNOLOGY**
ENGINEERING AND ENVIRONMENTAL SERVICES
SAINT LOUIS, COLLINSVILLE, KANSAS CITY

**UNCONFINED COMPRESSION TEST**
**ASTM D-2166**
**Stress-Strain Plot**

| | |
|---|---|
| Job No. | 0680801.32KC |
| Job Name | Black Snake Creek |
| Tested By | JB |
| Calculated By | yaw |
| Checked By | SDM |

| | | | |
|---|---|---|---|
| Max. Shear Strength = | 0.89 tsf | Water Content = | 23.2 % |
| Failure Strain = | 5.14 % | Dry Unit Weight = | 100.5 pcf |

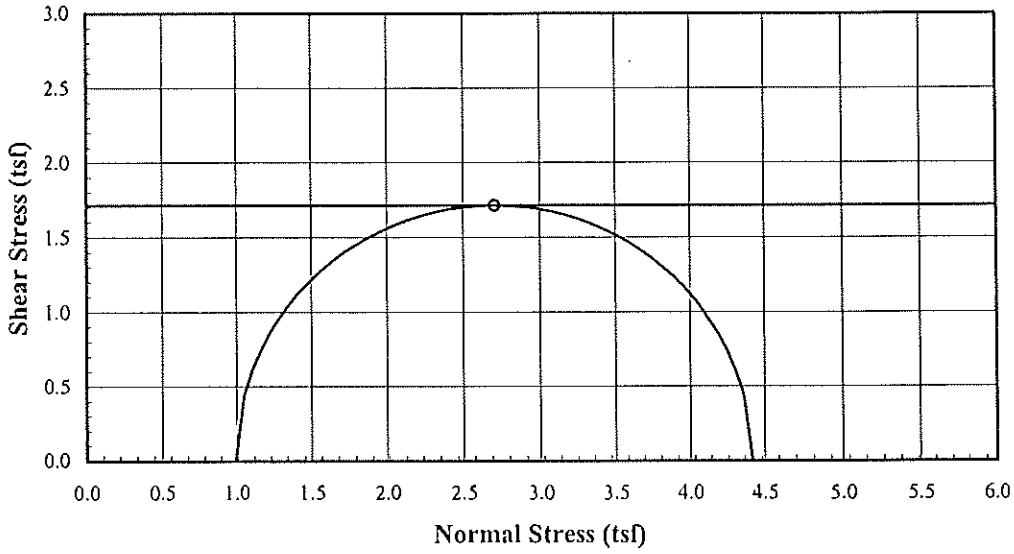


| | |
|---|---|
| Boring No. | ADU-1 |
| Sample No. | S-1 |
| Depth (ft) | 2.0-4.0' |
| Date | 8/13/04 |
| Sheet No. | |

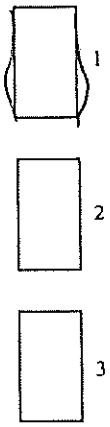
**GEOTECHNOLOGY**
ENGINEERING AND ENVIRONMENTAL SERVICES
SAINT LOUIS, COLLINSVILLE, KANSAS CITY

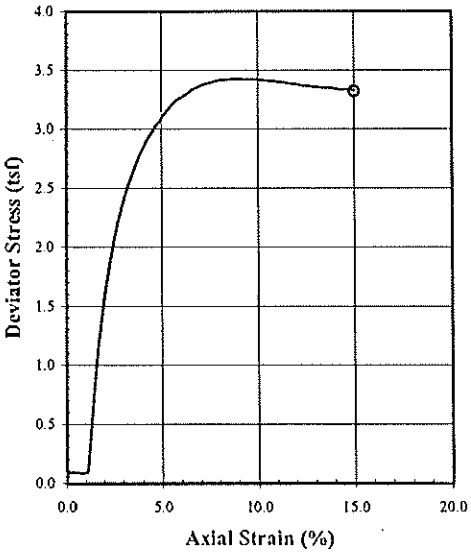
# UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST
## ASTM D2850

| | |
|---|---|
| Job No. | 0680801.32KM |
| Job Name | COE-Blacksnake Creek |
| Tested By | ARK |
| Calculated By | ARK |
| Checked By | yaw |




Failure Sketch




| | | Symbol | ○ | □ | △ | |
|---|---|---|---|---|---|---|
| | | Test No. | 1 | 2 | 3 | 4 |
| Initial | Water Content, % | $w_0$ | 16.3 | | | |
| | Dry Density, pcf | $\gamma_d$ | 100.6 | | | |
| | Void Ratio | $e_0$ | 0.66 | | | |
| | Saturation, % | $s_0$ | 65.9 | | | |
| Back Pressure | Water Content, % | $w_0$ | 16.3 | | | |
| | Dry Density, pcf | $\gamma_d$ | 100.6 | | | |
| | Void Ratio | $e_0$ | 0.66 | | | |
| | Saturation, % | $s_0$ | 65.9 | | | |
| | Final Back Pressure, psi | $u_0$ | 0 | | | |
| | Minor Prin. Stress, tsf | $\sigma_3$ | 0.99 | | | |
| | Max. Dev. Stress, tsf | $(\sigma_1 - \sigma_3)_{max}$ | 3.43 | | | |
| | Time to Failure, min | $t_f$ | | | | |
| | Rate of Strain, in/min | | 0.05 | | | |
| | Initial Diameter, in | $D_0$ | 2.791 | | | |
| | Initial Height, in | $H_0$ | 5.910 | | | |

Strain Controlled

Description of Specimens: Brown, low plasticity CLAY

| | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| LL= | 37 | | PL= | 22 | | PI= | 15 | | $G_s$= | 2.68 | Type of Speciment | Remolded @ 95% optima |

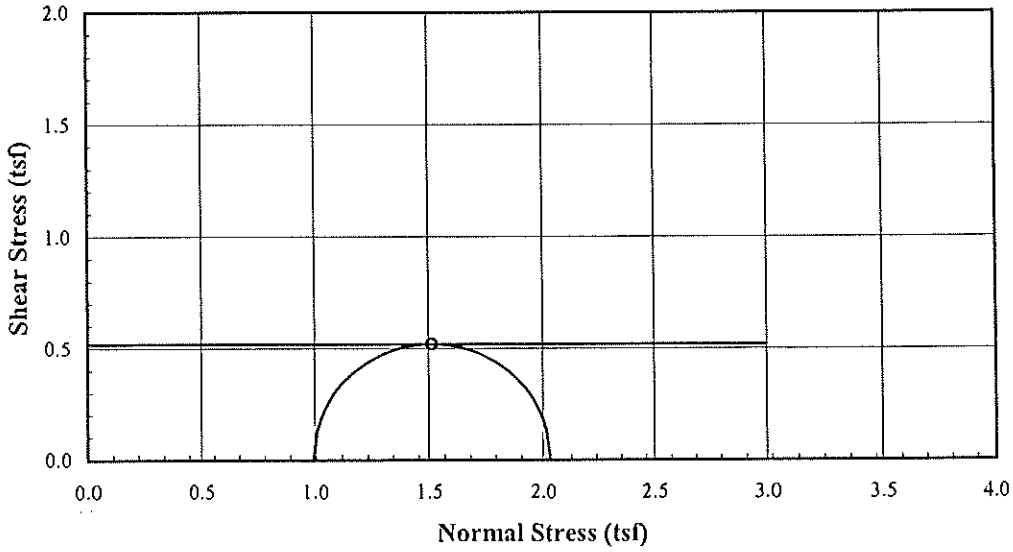
Remarks:

| | |
|---|---|
| Boring No. | ADU-3 |
| Sample No. | Wax-1 |
| Depth (ft) | 1.0-3.0' |
| Date | 8/15/04 |
| Sheet No. | |

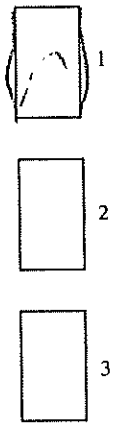
**GEOTECHNOLOGY**
ENGINEERING AND ENVIRONMENTAL SERVICES
SAINT LOUIS, COLLINSVILLE, KANSAS CITY

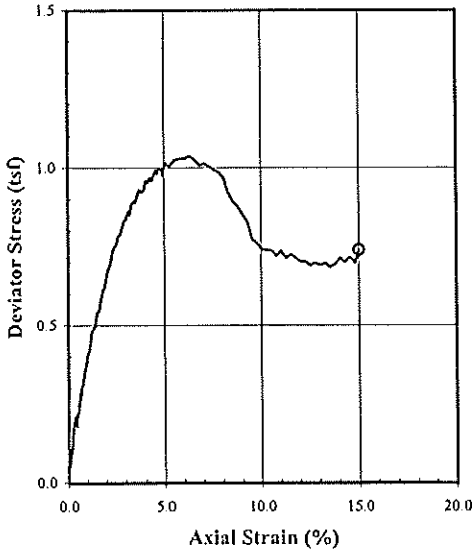
# UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST ASTM D2850

| | |
|---|---|
| Job No. | 0680801.32KM |
| Job Name | COE-Blacksnake Creek |
| Tested By | ARK |
| Calculated By | ARK |
| Checked By | yaw |




Failure Sketch




| | | Symbol | ○ | □ | △ | |
|---|---|---|---|---|---|---|
| | | Test No. | 1 | 2 | 3 | 4 |
| Initial | Water Content, % | $w_0$ | 20.6 | | | |
| | Dry Density, pcf | $\gamma_d$ | 93.9 | | | |
| | Void Ratio | $e_0$ | 0.78 | | | |
| | Saturation, % | $s_0$ | 70.7 | | | |
| Back Pressure | Water Content, % | $w_0$ | 20.6 | | | |
| | Dry Density, pcf | $\gamma_d$ | 93.9 | | | |
| | Void Ratio | $e_0$ | 0.78 | | | |
| | Saturation, % | $s_0$ | 70.7 | | | |
| | Final Back Pressure, psi | $u_0$ | 0 | | | |
| | Minor Prin. Stress, tsf | $\sigma_3$ | 0.99 | | | |
| | Max. Dev. Stress, tsf | $(\sigma_1 - \sigma_3)_{max}$ | 1.04 | | | |
| | Time to Failure, min | $t_f$ | | | | |
| | Rate of Strain, in/min | | 0.05 | | | |
| | Initial Diameter, in | $D_0$ | 1.473 | | | |
| | Initial Height, in | $H_0$ | 2.860 | | | |
| | | | | | | |

Strain Controlled

Description of Specimens: Dark brown, low plasticity CLAY

| | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| LL= | 38 | | PL= | 27 | | PI= | 11 | | $G_s$= | 2.68 | | Type of Speciment | 5" Wax Sample |

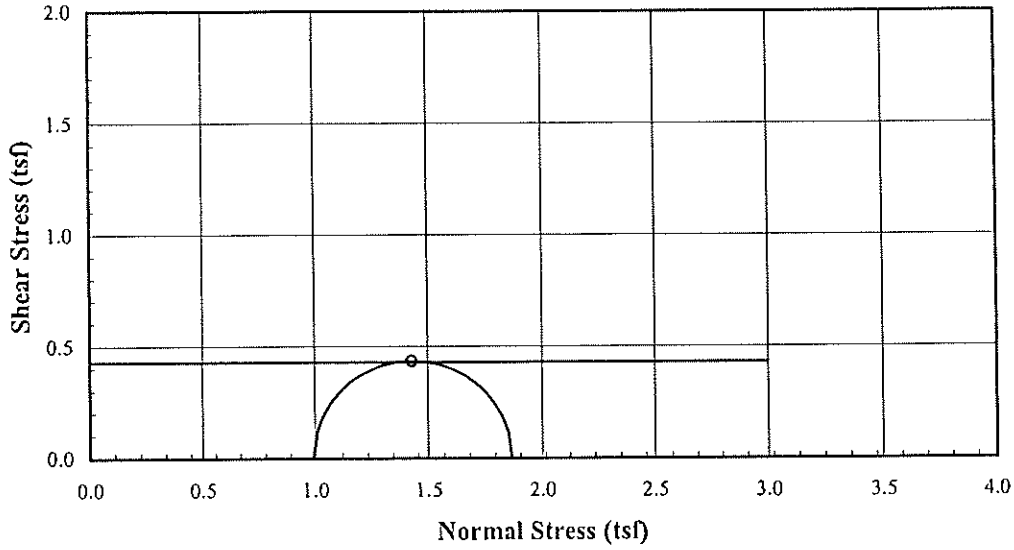
Remarks:

| | |
|---|---|
| Boring No. | ADU-5 |
| Sample No. | Wax-2 |
| Depth (ft) | 4.5-6.5' |
| Date | 8/13/04 |
| Sheet No. | |

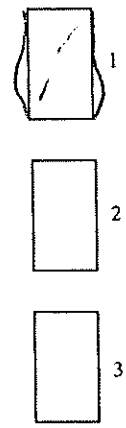
**GEOTECHNOLOGY**
ENGINEERING AND ENVIRONMENTAL SERVICES
SAINT LOUIS, COLLINSVILLE, KANSAS CITY

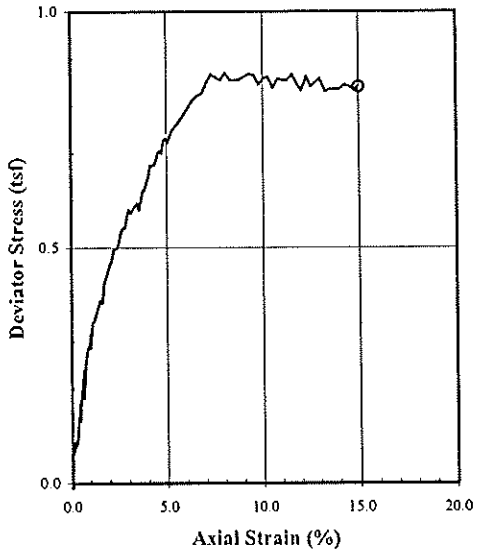
# UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST
## ASTM D2850

| | |
|---|---|
| Job No. | 0680801.32KM |
| Job Name | COE-Blacksnake Creek |
| Tested By | ARK |
| Calculated By | ARK |
| Checked By | yaw |




Failure Sketch




| | | Symbol | ○ | □ | △ | |
|---|---|---|---|---|---|---|
| | | Test No. | 1 | 2 | 3 | 4 |
| Initial | Water Content, % | $w_0$ | 26.8 | | | |
| | Dry Density, pcf | $\gamma_d$ | 96.5 | | | |
| | Void Ratio | $e_0$ | 0.73 | | | |
| | Saturation, % | $s_0$ | 97.9 | | | |
| Back Pressure | Water Content, % | $w_0$ | 26.8 | | | |
| | Dry Density, pcf | $\gamma_d$ | 96.5 | | | |
| | Void Ratio | $e_0$ | 0.73 | | | |
| | Saturation, % | $s_0$ | 97.9 | | | |
| | Final Back Pressure, psi | $u_0$ | 0 | | | |
| | Minor Prin. Stress, tsf | $\sigma_3$ | 0.99 | | | |
| | Max. Dev. Stress, tsf | $(\sigma_1 - \sigma_3)_{max}$ | 0.87 | | | |
| | Time to Failure, min | $t_f$ | | | | |
| | Rate of Strain, in/min | | 0.05 | | | |
| | Initial Diameter, in | $D_0$ | 1.472 | | | |
| | Initial Height, in | $H_0$ | 3.209 | | | |
| | | | | | | |

Strain Controlled

Description of Specimens: Dark brown, low plasticity CLAY

| | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| LL= | 40 | | PL= | 20 | | PI= | 20 | | $G_s$= | 2.68 | | Type of Speciment | 5" Wax Sample |

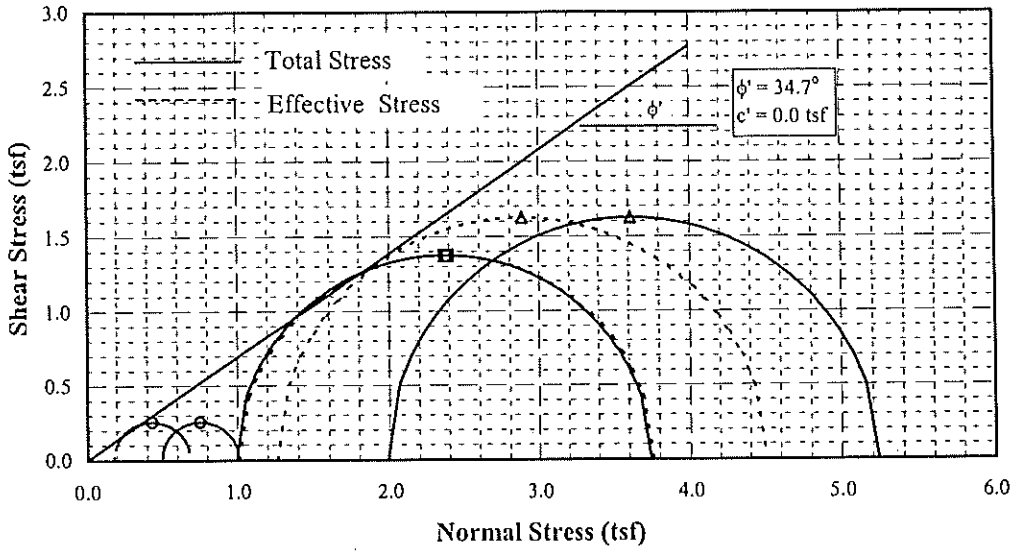
Remarks:

| | |
|---|---|
| Boring No. | ADU-1 |
| Sample No. | S-1 |
| Depth (ft) | |
| Date | 7/28/04 |
| Sheet No. | |

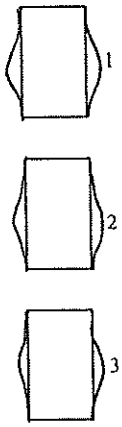
**GEOTECHNOLOGY**
ENGINEERING AND ENVIRONMENTAL SERVICES
SAINT LOUIS, COLLINSVILLE, KANSAS CITY

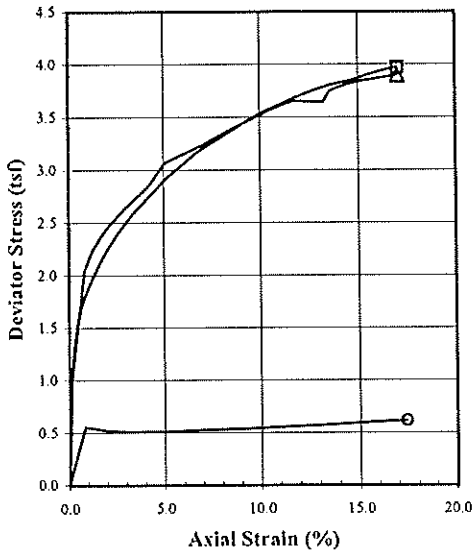
# CONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST
## ASTM D4767

| | |
|---|---|
| Job No. | 0680801.32KM |
| Job Name | Blacksnake Creek |
| Tested By | SR |
| Calculated By | yaw |
| Checked By | ARK |




Failure Sketch




| | | Symbol | ○ | □ | △ | |
|---|---|---|---|---|---|---|
| | | Test No. | 1 | 2 | 3 | |
| Initial | Water Content, % | $w_0$ | 16.0 | 16.0 | 16.0 | |
| | Dry Density, pcf | $\gamma_d$ | 101.4 | 101.5 | 101.5 | |
| | Void Ratio | $e_0$ | 0.67 | 0.67 | 0.67 | |
| | Saturation, % | $s_0$ | 64.7 | 64.8 | 64.8 | |
| Back Pressure | Water Content, % | $w_0$ | 23.1 | 22.6 | 22.0 | |
| | Dry Density, pcf | $\gamma_d$ | 104.0 | 104.9 | 106.0 | |
| | Void Ratio | $e_0$ | 0.63 | 0.62 | 0.60 | |
| | Saturation, % | $s_0$ | 99.3 | 99.2 | 99.7 | |
| | Final Back Pressure, psi | $u_0$ | 35.8 | 35.2 | 35.3 | |
| | Minor Prin. Stress, tsf | $\sigma_3$ | 0.50 | 0.99 | 1.99 | |
| | Max. Dev. Stress, tsf | $(\sigma_1 - \sigma_3)_{max}$ | 0.51 | 2.75 | 3.25 | |
| | Time to Failure, min | $t_f$ | | | | |
| | Rate of Strain, in/min | | 0.01 | 0.01 | 0.01 | |
| | Initial Diameter, in | $D_0$ | 2.823 | 2.823 | 2.823 | |
| | Initial Height, in | $H_0$ | 5.945 | 5.945 | 5.945 | |
| | B-Value | | 0.98 | 0.99 | 0.99 | |

Strain Controlled

Description of Specimens: Brown, low plasticity CLAY

| | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| LL= | 37 | | PL= | 22 | | PI= | 15 | | $G_s$= | 2.72 | | Type of Speciment | Remolded Sample |

Remarks:

| | |
|---|---|
| Boring No. | ADU-3 |
| Sample No. | Wax-1 |
| Depth (ft) | 1'-3' |
| Date | 8/11/04 |
| Sheet No. | |

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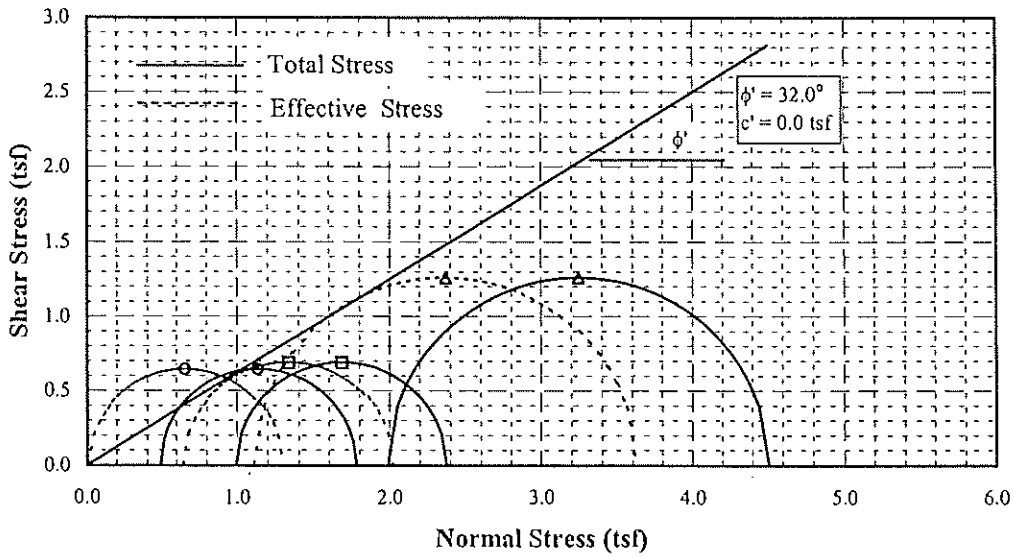
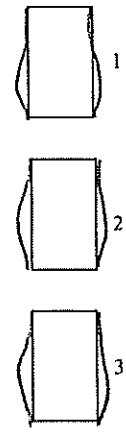
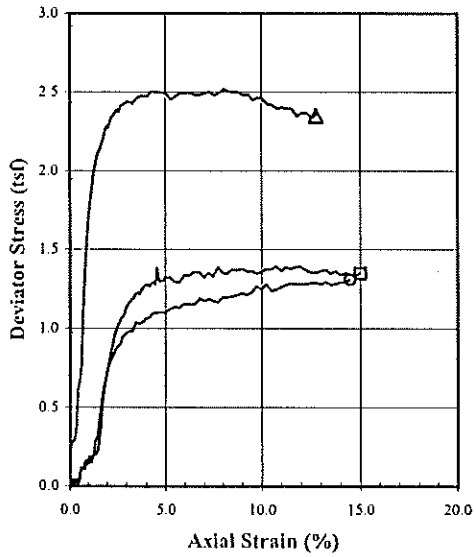
# CONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST
## ASTM D4767

| | |
|---|---|
| Job No. | 0680801.32KM |
| Job Name | COE-Blacksnake Creek |
| Tested By | ARK |
| Calculated By | ARK |
| Checked By | yaw |




Failure Sketch




| | | Symbol | ○ | □ | △ | |
|---|---|---|---|---|---|---|
| | | Test No. | 1 | 2 | 3 | |
| Initial | Water Content, % | $w_0$ | 21.8 | 19.8 | 20.0 | |
| | Dry Density, pcf | $\gamma_d$ | 91.3 | 92.5 | 92.8 | |
| | Void Ratio | $e_0$ | 0.83 | 0.81 | 0.80 | |
| | Saturation, % | $s_0$ | 70.5 | 65.6 | 66.9 | |
| Back Pressure | Water Content, % | $w_0$ | 29.3 | 27.0 | 26.2 | |
| | Dry Density, pcf | $\gamma_d$ | 93.6 | 97.0 | 98.2 | |
| | Void Ratio | $e_0$ | 0.79 | 0.72 | 0.70 | |
| | Saturation, % | $s_0$ | 100.0 | 99.8 | 99.8 | |
| | Final Back Pressure, psi | $u_0$ | 35.1 | 35.7 | 31.8 | |
| | Minor Prin. Stress, tsf | $\sigma_3$ | 0.49 | 0.99 | 1.99 | |
| | Max. Dev. Stress, tsf | $(\sigma_1 - \sigma_3)_{max}$ | 1.29 | 1.38 | 2.52 | |
| | Time to Failure, min | $t_f$ | | | | |
| | Rate of Strain, in/min | | 0.01 | 0.01 | 0.01 | |
| | Initial Diameter, in | $D_0$ | 1.491 | 1.481 | 1.486 | |
| | Initial Height, in | $H_0$ | 3.001 | 3.022 | 2.954 | |
| | B-Value | | 0.99 | 0.98 | 0.98 | |

Strain Controlled

Description of Specimens: Brown, low plasticity CLAY

| | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| LL= | 38 | | PL= | 27 | | PI= | 11 | | $G_s$= | 2.68 | | Type of Speciment | 5" Wax Sample |

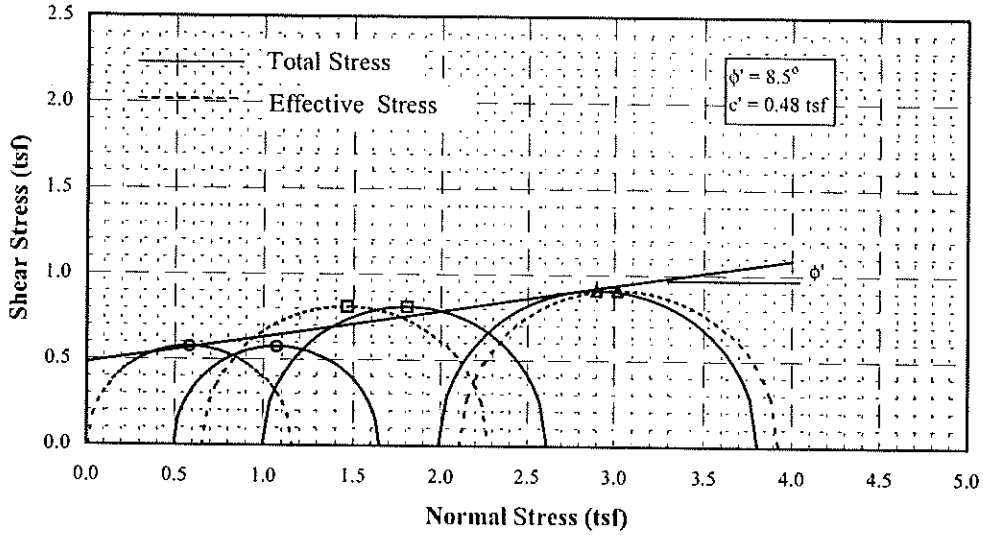
Remarks:

| | |
|---|---|
| Boring No. | ADU-5 |
| Sample No. | Wax-2 |
| Depth (ft) | 4.5-6.5' |
| Date | 8/13/04 |
| Sheet No. | |

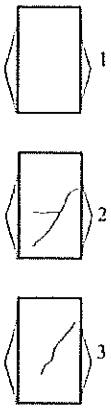
**GEOTECHNOLOGY**
ENGINEERING AND ENVIRONMENTAL SERVICES
SAINT LOUIS, COLLINSVILLE, KANSAS CITY

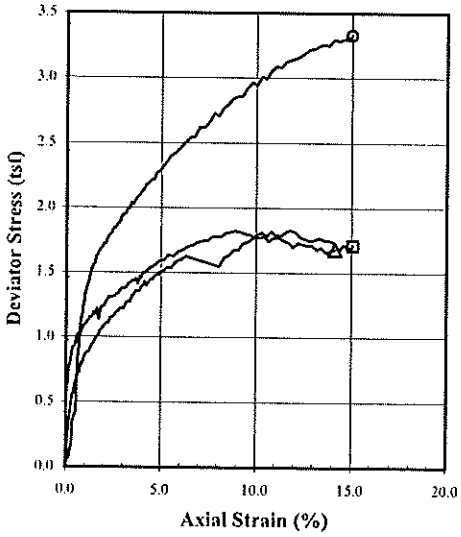
**CONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST**
**ASTM D4767**

| | |
|---|---|
| Job No. | 0680801.32KM |
| Job Name | COE-Blacksnake Creek |
| Tested By | ARK |
| Calculated By | ARK |
| Checked By | yaw |




Failure Sketch




| | | Symbol | ○ | □ | △ | |
|---|---|---|---|---|---|---|
| | | Test No. | 1 | 2 | 3 | |
| Initial | Water Content, % | $w_0$ | 27.7 | 27.3 | 27.0 | |
| | Dry Density, pcf | $\gamma_d$ | 95.9 | 96.1 | 95.7 | |
| | Void Ratio | $e_0$ | 0.74 | 0.74 | 0.75 | |
| | Saturation, % | $s_0$ | 99.7 | 99.0 | 96.6 | |
| Back Pressure | Water Content, % | $w_0$ | 27.8 | 27.4 | 26.9 | |
| | Dry Density, pcf | $\gamma_d$ | 95.9 | 96.4 | 97.2 | |
| | Void Ratio | $e_0$ | 0.74 | 0.73 | 0.72 | |
| | Saturation, % | $s_0$ | 100.2 | 99.9 | 99.9 | |
| | Final Back Pressure, psi | $u_0$ | 30 | 40.9 | 30 | |
| | Minor Prin. Stress, tsf | $\sigma_3$ | 0.50 | 0.99 | 1.99 | |
| | Max. Dev. Stress, tsf | $(\sigma_1 - \sigma_3)_{max}$ | 1.15 | 1.62 | 1.82 | |
| | Time to Failure, min | $t_f$ | | | | |
| | Rate of Strain, in/min | | 0.025 | 0.025 | 0.025 | |
| | Initial Diameter, in | $D_0$ | 1.483 | 1.462 | 1.479 | |
| | Initial Height, in | $H_0$ | 3.204 | 3.227 | 3.201 | |
| | B-Value | | 0.97 | 0.97 | 0.95 | |

Strain Controlled

Description of Specimens: Dark brown, low plasticity CLAY

| | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| LL= | 40 | | PL= | 20 | | PI= | 20 | | $G_s$= | 2.68 | | Type of Speciment | 5" Wax Sample |

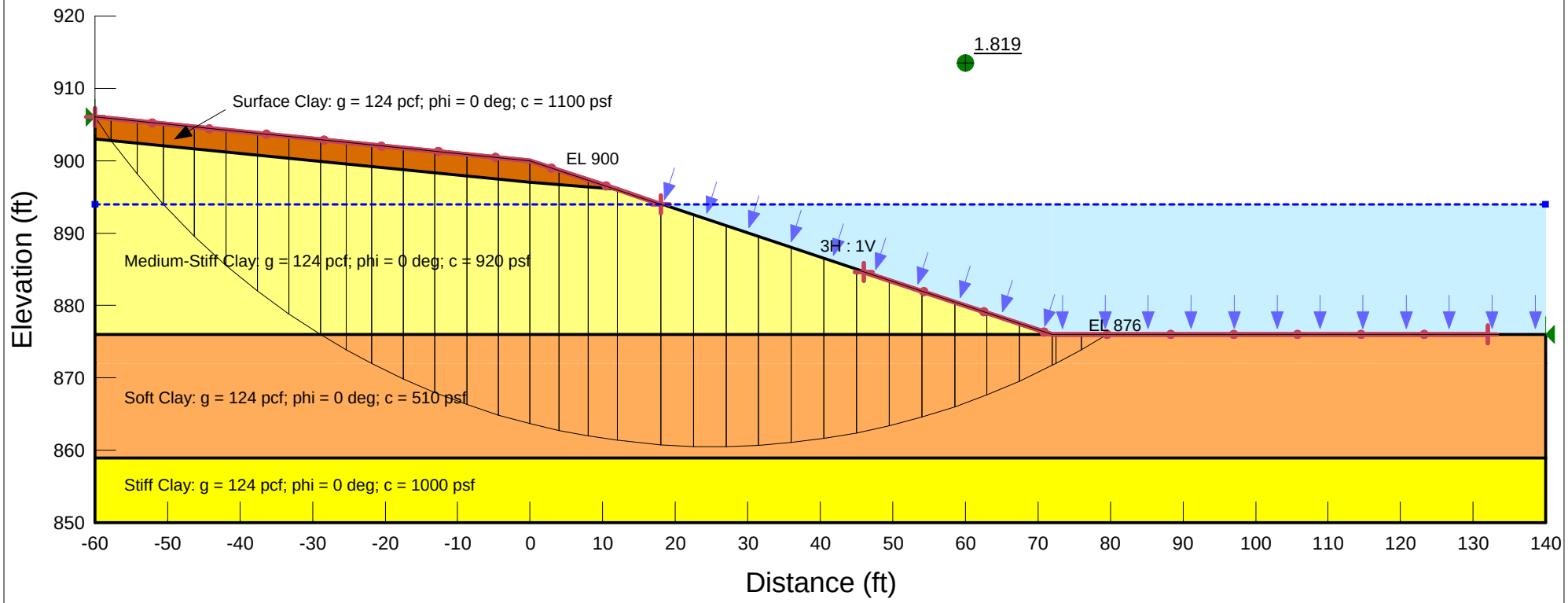
Remarks:

# Appendix B

# Stability Computations

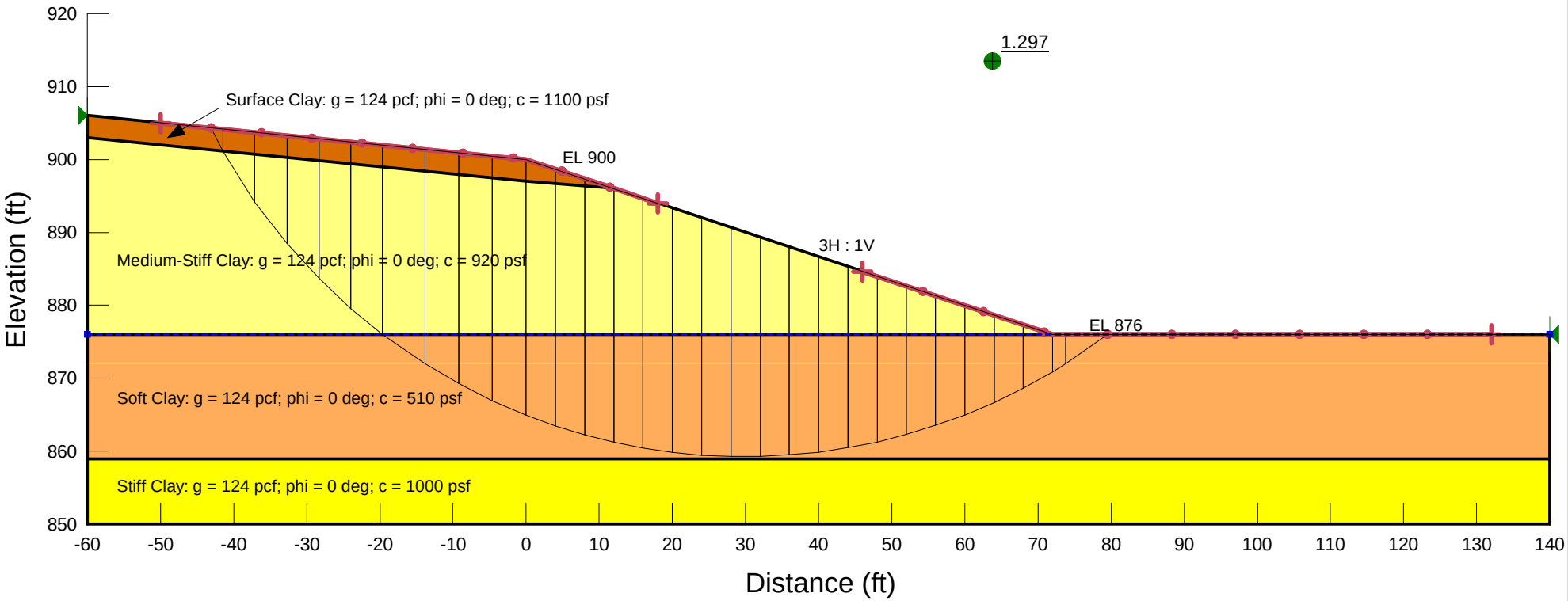
# Blacksnake Creek
# Excavation Stability Analysis
# Total Stress Condition 1

Filename: Blacksnake_02c.gsz
Computed: JRJ Date: 12/10/2012
Checked: LDK Date: 12/20/2012



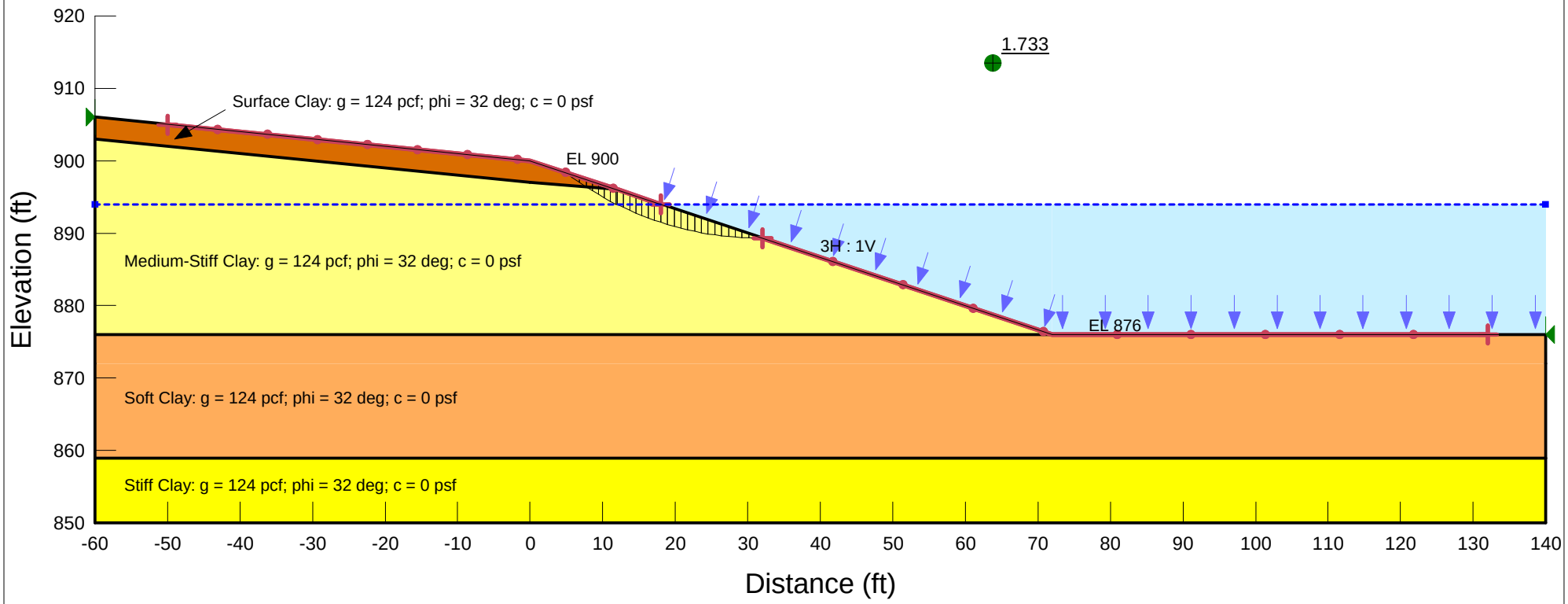
# Blacksnake Creek
# Excavation Stability Analysis
# Total Stress Condition 2

Filename: Blacksnake_02b.gsz
Computed: JRJ Date: 12/10/2012
Checked: LDK Date: 12/20/2012



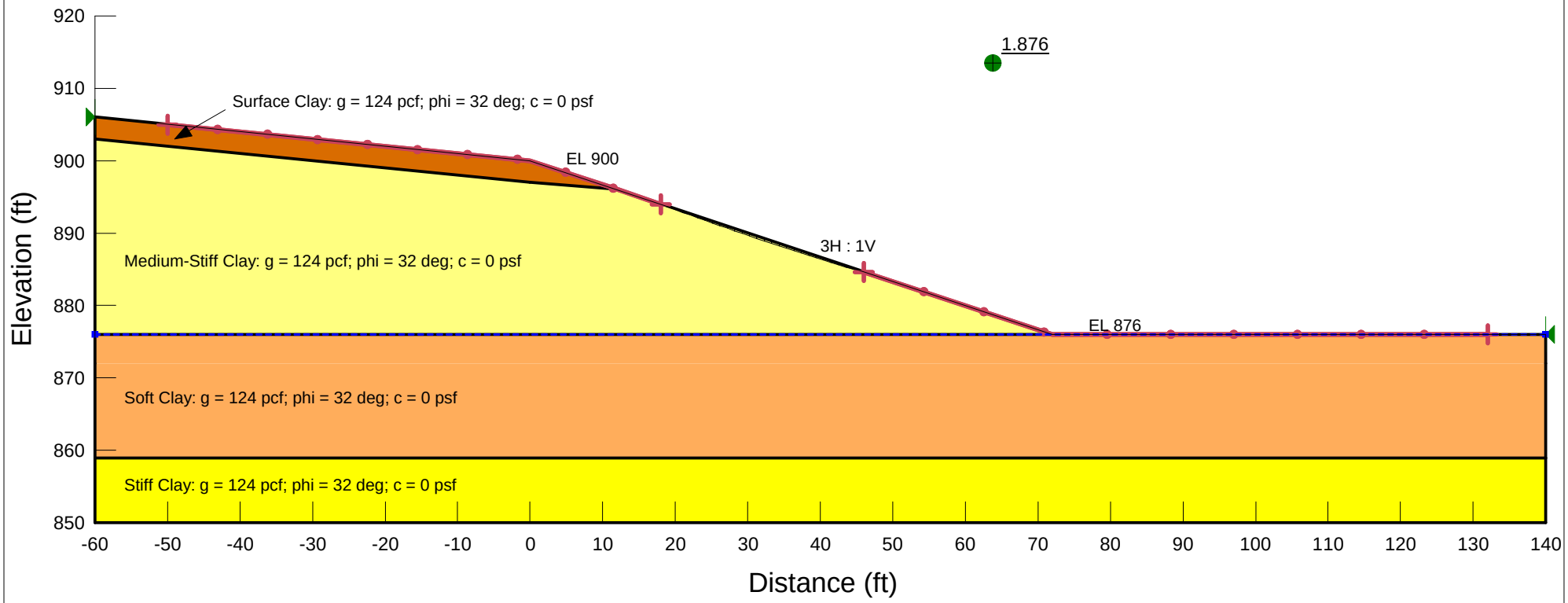
# Blacksnake Creek
# Excavation Stability Analysis
# Effective Stress Condition 1

Filename: Blacksnake_03b.gsz
Computed: JRJ Date: 12/10/2012
Checked: LDK Date: 12/20/2012



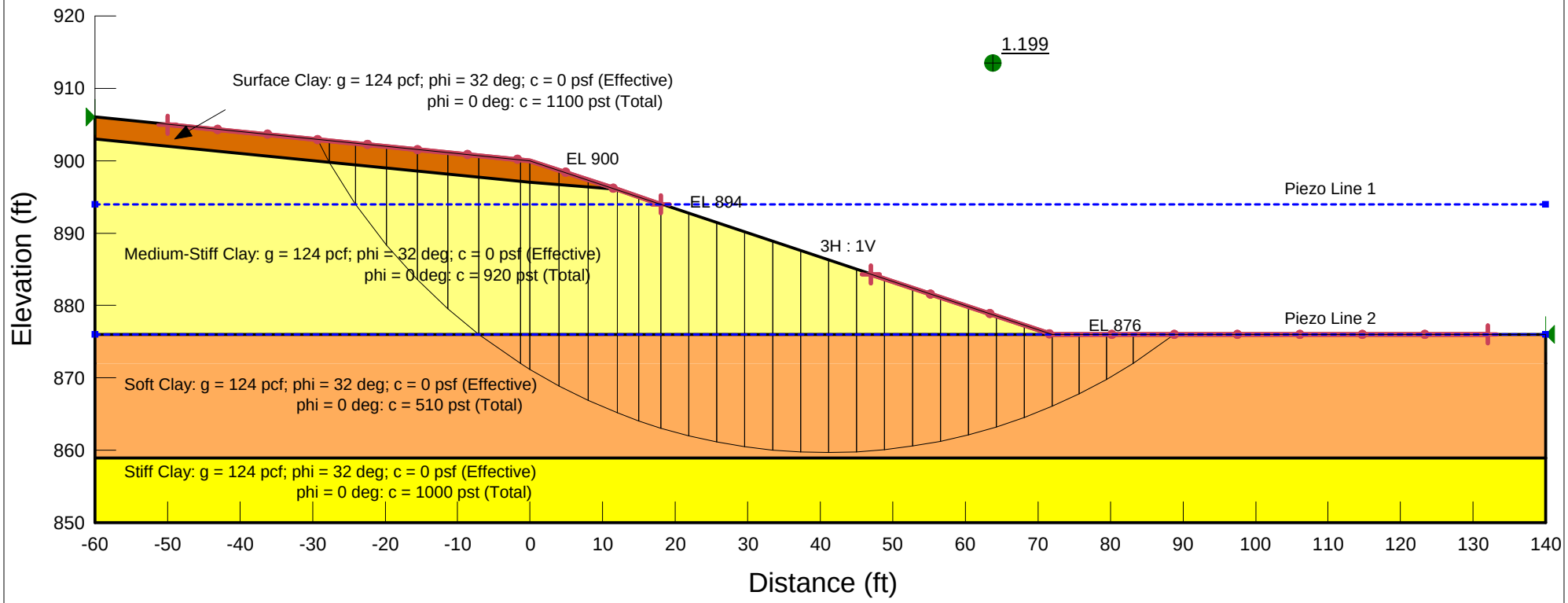
# Blacksnake Creek
# Excavation Stability Analysis
# Effective Stress Condition 2

Filename: Blacksnake_03a.gsz
Computed: JRJ Date: 12/10/2012
Checked: LDK Date: 12/20/2012



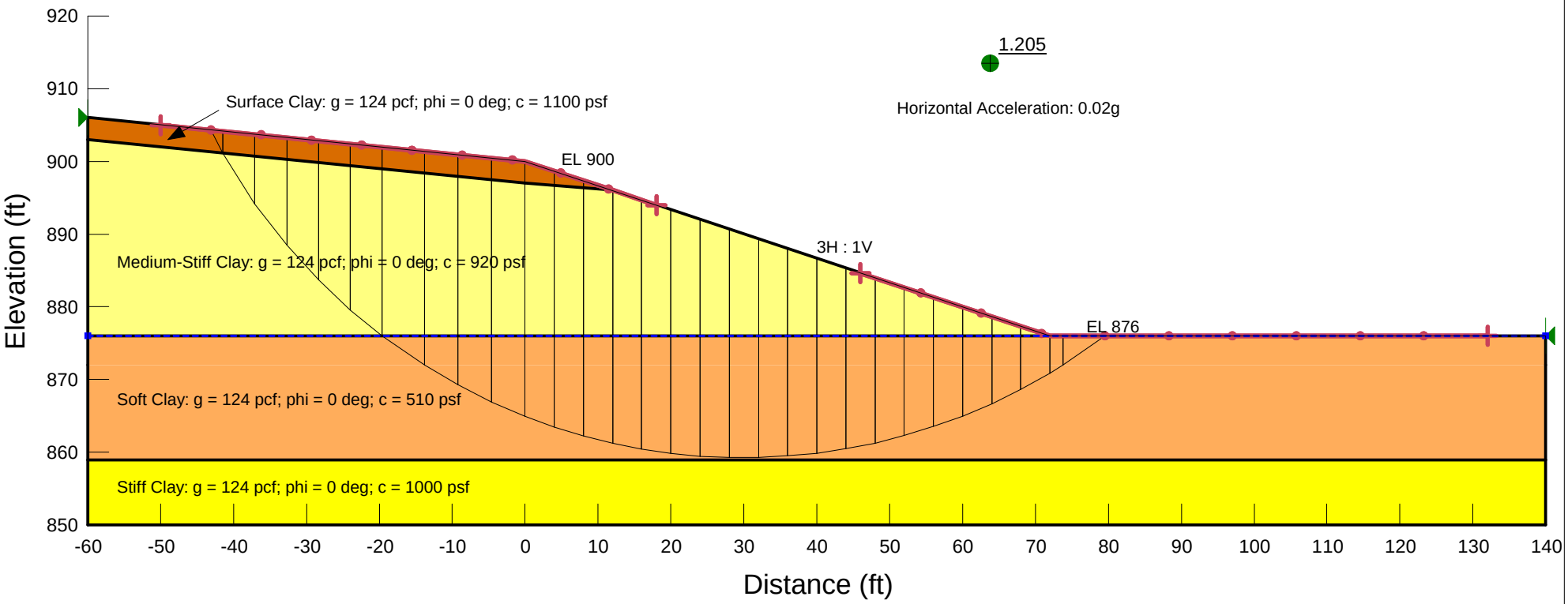
# Blacksnake Creek
# Excavation Stability Analysis
# Rapid Drawdown Condition

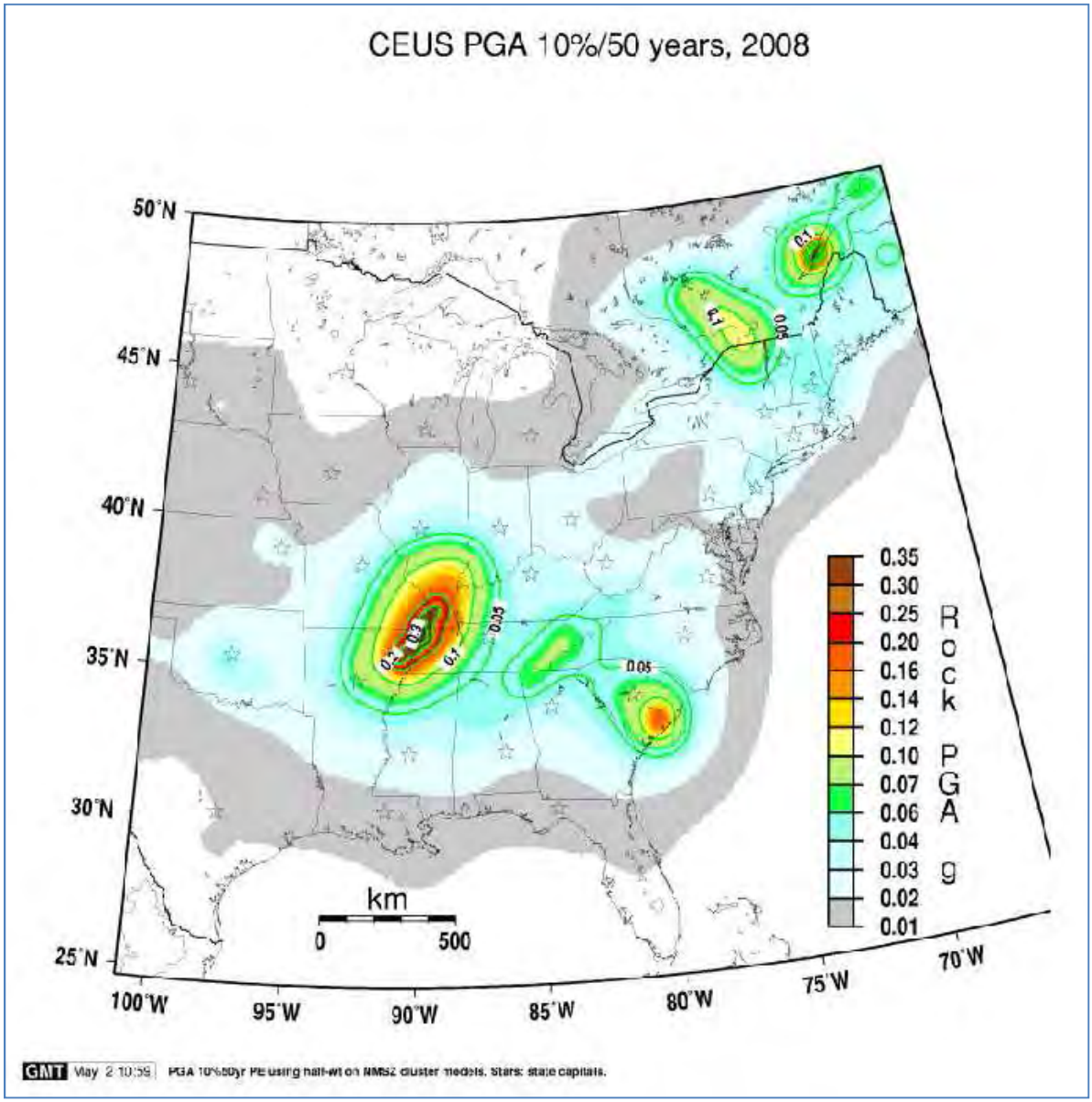
Filename: Blacksnake_04 - Stage Drawdown.gsz
Computed: JRJ Date: 12/10/2012
Checked: LDK Date: 12/20/2012



# Blacksnake Creek
# Excavation Stability Analysis
# Total Stress Condition - Seismic Analysis

Filename: Blacksnake_02f.gsz
Computed: JRJ Date: 12/10/2012
Checked: LDK Date: 12/20/2012




CEUS PGA 10%/50 years, 2008
50°N
45°N
40°N
35°N
30°N
25°N
100°W
95°W
90°W
85°W
80°W
75°W
70°W
0.1
0.05
0.3
0.2
0.1
0.05
0.05
km
0
500
0.35
0.30
0.25
0.20
0.16
0.14
0.12
0.10
0.07
0.06
0.04
0.03
0.02
0.01
Rock PGA g
GMT May 2 10:59
PGA 10%50yr PE using half-wt on NMS2 cluster models. Stars: state capitals.

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Project No. _____ Page No. _____

Subject Black Snake Creek

Computed by JRJ Date 12-10-12

Checked by _____ Date _____

Approved by _____ Date _____

Total Stress Parameters

Sheet No. _____ of _____

SURFACE CLAY: 0-3'

ADU-5 c = 1460 psf (UC)
ADU-3 c = 1040 psf (UU)
ADU-6 c = 1950 psf (UC)

} $\frac{2}{3}$ RULE: Assume c = 1100 psf

MEDIUM-STIFF CLAY: 3' - 24'

ADU-5: ~~c = 870 psf (UU)~~ * inconsistent test data sheet
c = 1080 psf (UC)
c = 970 psf (UC)
c = 676 psf (TORVANE)

ADU-3: c = 819 psf (TORVANE)
c = 1331 psf (TORVANE)
c = 1024 psf (TORVANE)

ADU-6: c = 1126 psf (TORVANE)
c = 890 psf (UC)
c = 819 psf (TORVANE)

Assume c = 920 psf ($\frac{2}{3}$ of UC Test Data and $\frac{1}{3}$ of TORVANE Test Data at or above assumed parameter)

AD-4: N = 21, 14 } N = 14 ⇒ c = 1700 psf

N = 5, 6, 6, 3 } N = 5 ⇒ c = 500 psf (EM 1110-2-2504 Table 3-4)

→ THIS IS LIKELY PART OF SOFT CLAY STRATUM.

*Med-Stiff clay stratum is thinner or boring elevation is lower.

SOFT CLAY: 24' - 41'

ADU-5: c = 410 psf
c = 512 psf
c = 614 psf
c = 307 psf
c = 717 psf
} TORVANE.

ADU-6: c = 2048 psf (TORVANE) — LIKELY PART OF STIFF CLAY STRATUM.

Assume: c = 510 psf

SC 3004 R4 898

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Project No. _____ Page No. _____

Subject BLACK SNAKE CREEK

Computed by JRS Date 12-10-12

Checked by _____ Date _____

Approved by _____ Date _____

Sheet No. _____ of _____

STIFF CLAY:

ADU-5: c = 1126 psf

c = 1229 psf

c = 717 psf (SANDY)

c = 614 psf (SANDY)

c = 410 psf (SANDY)

ASSUME: c = 1,000 psf

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Project No. _____ Page No. _____

Subject BLACK SNAKE CREEK

Computed by JRS Date 12-10-12

Checked by _____ Date _____

EFFECTIVE STRESS PARAMETERS

Approved by _____ Date _____

Sheet No. _____ of _____

EFFECTIVE STRESS PARAMETERS BASED ON ATTERBERG LIMIT CORRELATIONS.

Typical PI RANGE : 10 → 20

This corresponds to $\phi' = 31° - 33°$

Assume $\phi = 32°$ for all strata.

TRIAXIAL Testing lab data exhibited inconsistencies. This data was ignored.

SC 3004 R4 898

EFFECTIVE STRESS



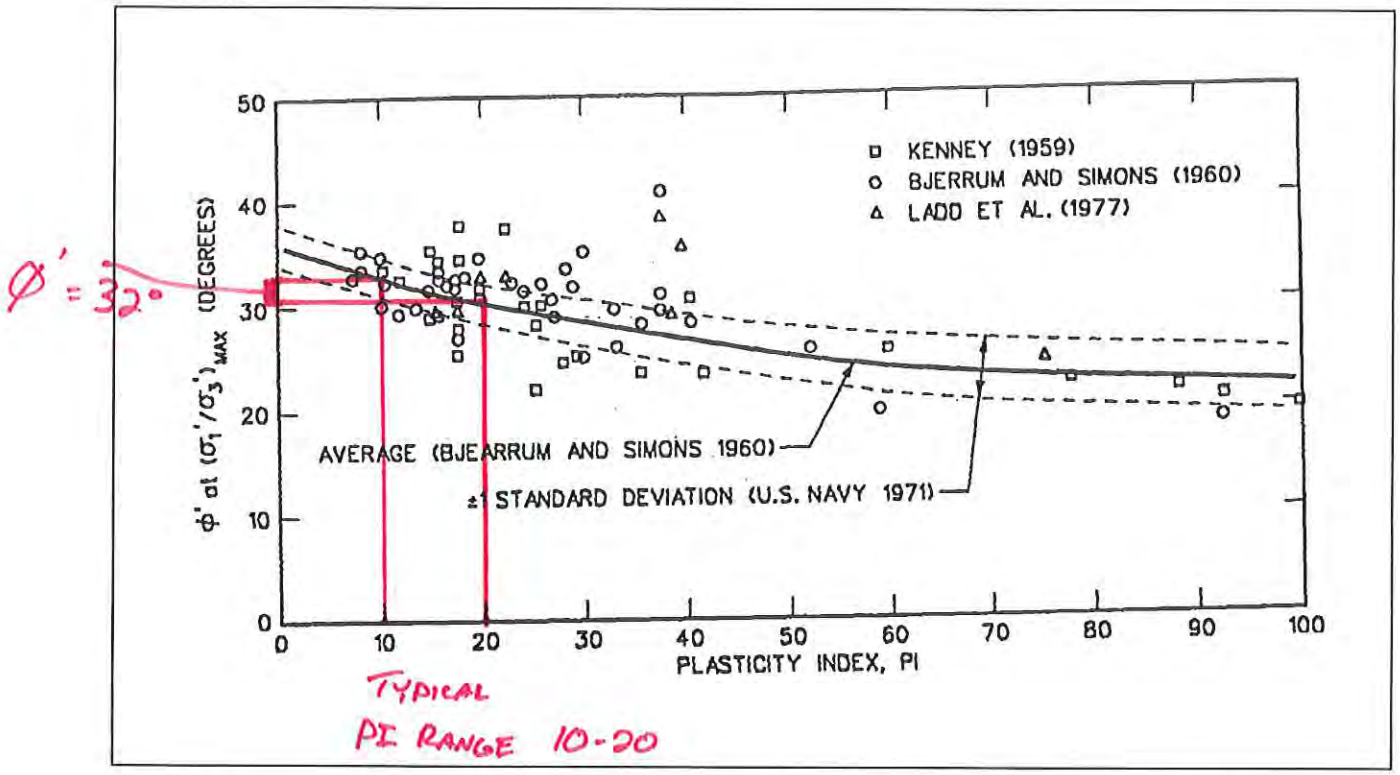


**Figure 3-4. Empirical correlation between friction angle and PI from triaxial tests on normally consolidated clays**

stress of 1 ton per square foot using an equation of the form:

$$N' = C_N N \tag{3-1}$$

where

$N'$ = corrected resistance

$C_N$ = correction factor

$N$ = measured resistance

Table 3-5 and Figure 3-5 summarize the some most commonly proposed values for $C_N$. Whitman and Liao (1984) developed the following expression for $C_N$:

$$C_N = \sqrt{\frac{1}{\sigma'_{vo}}} \tag{3-2}$$

where effective stress due to overburden, $\sigma'_{vo}$, is expressed in tons per square foot. The drained friction angle $\phi'$ can be estimated from $N'$ using Figure 3-6. The relative density of normally consolidated sands can be estimated from the correlation obtained by Marcuson and Bieganousky (1977):

$$D_r = 11.7 + 0.76[|222(N) + 1600 - 53(p'_{vo}) - 50(C_u)^2|]^{1/2} \tag{3-3}$$

where

$p'_{vo}$ = effective overburden pressure in pounds per square inch

$C_u$ = coefficient of uniformity ($D_{60}/D_{10}$)

Correlations have also been proposed between the SPT and the undrained strength of clays (see Table 3-4). However, these are generally unreliable and should be used for very preliminary studies only and for checking the reasonableness of SPT and lab data.

*c. Cone penetration test.* The CPT (ASTM D 3441-79 (1986a)) is widely used in Europe and is gaining

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| | | | |
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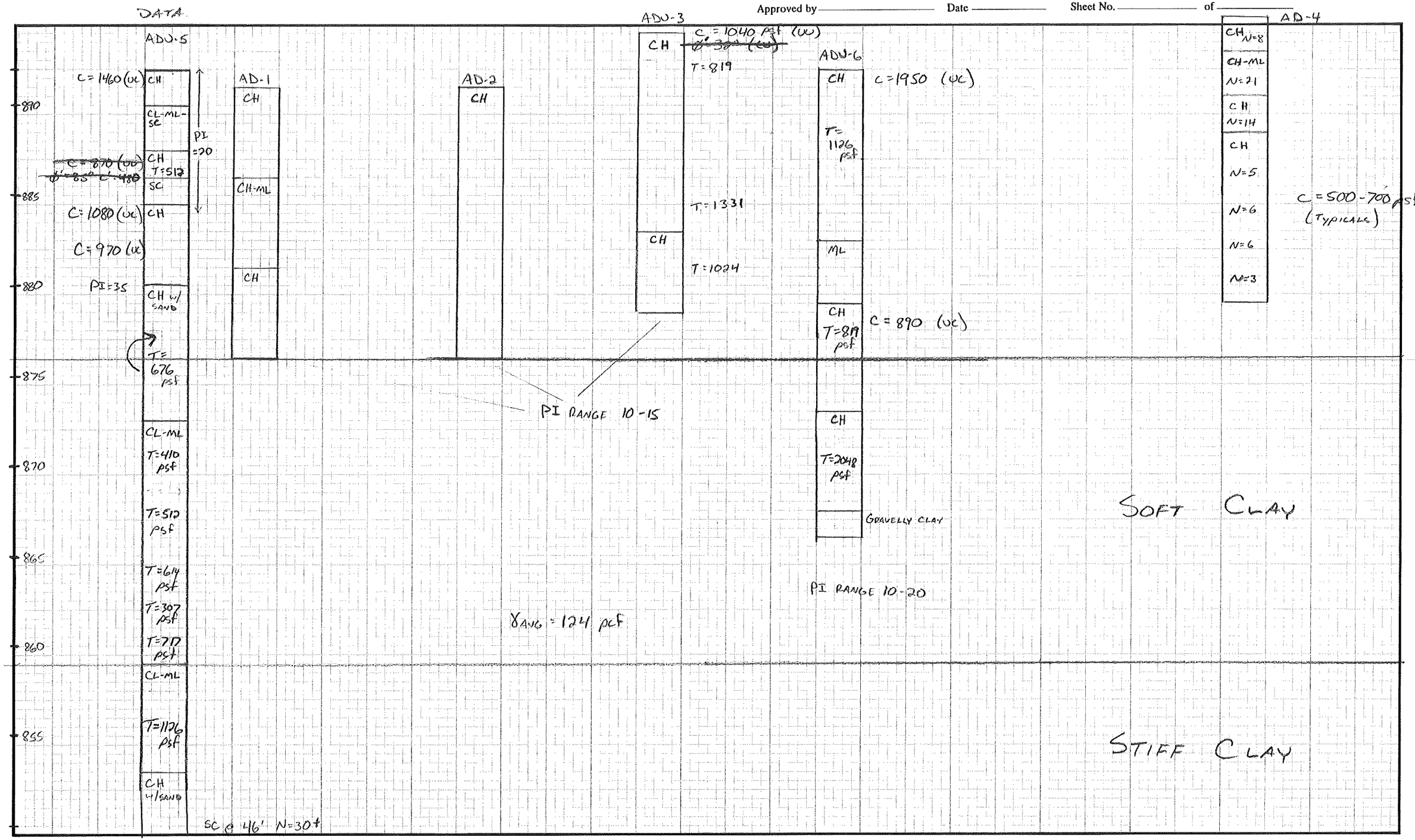
* ELEVATIONS NOT INCLUDED ON SOIL BORINGS
ELEVATIONS ARE APPROXIMATED BASED ON TOPOGRAPHIC DATA.




SC 3006

# Appendix C

# Erosion Protection Calculations

Stanley Consultants INC.

Project No. 23842 Page No. 1

Subject BLACKSNAKE CREEK
OVERFLOW SPILLWAY RIPRAP SIZING
@ NORTHWEST PARKWAY

Computed by EJD Date 8/27/12
Checked by JRJ Date 1-11-13
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METRIC UNITS

D/S SIDE SLOPE = 1V : ~~6~~ 8 H  FITS TOPO BEST

WEIR LENGTH = 249' @ ELEV 898.7 ($Q_{100}$)  75.90 m

HEIGHT OVER WEIR = 898.7 − 896.3 = ~~2.2'~~ 2.4  0.67 m

↑ FROM CAD FILE PROFILE

OVERTOPPING FLOW FROM WEIR EQN = $Q = CLH^{1.5}$

$C \cong 2.6$

$Q = (2.6)(249)(2.4)^{1.5} = \cancel{2113}$ cfs 2407  60 m³/s

$q$ = FLOW PER UNIT LENGTH  $q = \frac{2113 \text{ cfs}}{249 \text{ ft}} = \cancel{8.49}$ cfs/ft 9.67  ~~0.788~~ .898 m³/s / m

RIPRAP SIZING
EQUATIONS PER U.S.B.R "SIMPLIFIED DESIGN GUIDE FOR RIPRAP SUBJECTED TO OVERTOPPING FLOW"
PAP-0790, 1998  (EQUATIONS ARE IN METRIC UNITS)

FROM FIGURE 4 ⇒ $D_{50} C_U^{0.25} = \cancel{0.23}$ 0.20

$S = \cancel{0.17}$ 0.125

$C_U = 1.95$ COEF. OF UNIFORMITY

$D_{50} = \frac{\cancel{0.23}\ .20}{(1.95)^{.25}} = \cancel{0.195}$ m  0.169

≅ ROUND TO 0.20 m FOR ITERATION 1

ITERATION #1

INTERSTITIAL VELOCITY

$V_i = 2.48\, C_U^{-2.22}\, S^{0.58} \sqrt{g(D_{50})}$

$= 2.48\,(1.95)^{-2.22}(\cancel{0.17}\ 0.125)^{.58}\sqrt{(9.81)(0.2)}$

$= \cancel{0.282}$ m/s  0.236 m/s

$V_{AVE} = V_i \times n_p = (\cancel{0.282}\ .236 \text{ m/s})(.45) = \cancel{0.127}\ 0.106$ m/s

↑ POROSITY = 0.45

$Y$ = AVG DEPTH OF WATER AT POINT OF INCIPIENT FAILURE $= q / V_{AVE} = \frac{0.788}{\cancel{0.127}\ .106} = \cancel{6.20}\ 7.42$ m

$Y > 2D_{50}$

CONTINUE THRU STEPS 5-10

Stanley Consultants INC.

Project No. 23842 Page No. 2

Subject BLACKSNAKE CREEK OVERFLOW SPILLWAY RIPRAP SIZING @ NORTHWEST PARKWAY

Computed by ADP Date 8/27/12

Checked by JRS Date 1-11-13

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$$h = \frac{(0.06)(2.65-1.00)(.20)(0.900)}{0.97\ (0.125)} = 0.147$$

$$n = 0.0414\,(D_{50})^{1/6} = 0.0414\,(0.20)^{.17} = 0.0315$$

$$q_1 = \frac{1}{(.0315)}(.147)^{1.67}(.125)^{.5} = 0.456$$

$$q_2 = .\cancel{788}\ .898 - .456 = 0.\cancel{33}\ .442$$

$$h_2 = \frac{0.\cancel{33}\ .442}{0.106} = \cancel{3.13}\ 4.17 \Rightarrow h_2 > 4D_{50}$$

∴ INCREASE $D_{50}$ SIZE AND GO TO ITERATION #2

EXCEL SPREADSHEET ITERATIONS FOR $D_{50}$

YIELD $D_{50} = 0.28$ m WHICH MEETS CRITERIA

$= 0.92$ FT $= 11''$

THICKNESS SHALL BE BETWEEN $2D_{50}$ & $4D_{50}$

$2D_{50} = 22''$ $3D_{50} = 33''$ $4D_{50} = 44''$

▷ ADD SAFETY FACTOR, BUMP UP TO MODOT - TYPE IV ⇒ $D_{50} = 19''$

MAX SIZE = 28''

$2D_{50} = 38'' = 3.2'$

$3D_{50} = 4.75'$

CHECK $1.5\,D_{100} = 42'' = 3.5'$ ✓

USE 3.5' THICKNESS ✓

SC 3020 R2 199

Stanley Consultants INC.

Project No. 23842 Page No.

Subject BLACKSNAKE CREEK OVERFLOW SPILLWAY RIPRAP SIZING @ NORTHWEST PARKWAY

Computed by GD Date 12/6/12

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CHECK RIPRAP SIZING FOR SPILLWAYS

EQUATIONS FOUND IN NRCS STREAM RESTORATION DESIGN HANDBOOK TS-14C

ABT & JOHNSON METHOD

$$D_{50} = (q_{design})^{0.56} \times S^{0.43} \times 5.23 \qquad (TS14C-14)$$

$$= (8.49\ cfs/ft)^{.56} (0.125)^{0.43} (5.23)$$

$$= 7.09 \text{ INCHES}$$

ARS ROCK CHUTES

$$D_{50} = 12\,(0.233 q S^{0.58})^{.529} \qquad (TS14C-17)$$

$$= 12\left[(0.233)(8.49)(.125)^{.58}\right]^{.529}$$

$$= 9.10 \text{ INCHES}$$

USACE STEEP SLOPE RIPRAP (TS14C-12)

$$D_{30} = \frac{1.95\, S^{0.555} (Cq)^{.667}}{g^{.333}} = \frac{1.95\,(.125)^{.555}(1.25 \times \cancel{8.49}\ 9.67)^{.667}}{(32.2)^{.333}}$$

$$= \cancel{0.934'}\ 1.02'$$

$$D_{50} \cong \cancel{1.07'}\ 1.17' = \cancel{12.89}\ 14.08 \text{ INCHES} \quad ✓$$

MOST CONSERVATIVE

WITH $\cancel{1.4}$ 1.3 SAFETY FACTOR $\Rightarrow D_{50} = 18$ INCHES ✓

CORRESPONDS TO MO-DOT TYPE IV ROCK DITCH LINER ($D_{50} \cong 19''$)

$$\text{THICKNESS} = 1.5 \times D_{100} = 1.5 \times 28'' = 42''$$

$$= \underline{3.5' \text{ THICKNESS}} \quad ✓$$

SC 3020 R2 199

***Design Procedure to Predict Stable Stone Size***

A new design procedure to predict median stone size for a protective riprap layer has been developed from the test program and compilation of data from previous investigations [2,4,5,6]. A set of curves shown in Figure 4 for different embankment slopes combines the rock properties of the riprap material, discharge, and embankment slope. Each curve represents the point of incipient failure for a particular embankment slope, S, for a design unit discharge, q, and median stone size, $D_{50}$. $C_u$ on the y-axis is the coefficient of uniformity of the material which is the ratio of the material $D_{60}$ to $D_{10}$. The curves on figure 4 are based on the riprap material having an angle of internal friction, Φ, of 42°. The design curves combine empirical data with accepted sediment transport equations and are not simply a best fit of the data. A safety factor is not included in the graph, but left to the judgement of the designer to apply as needed.

Further investigation of the data used to determine these design curves can lead to some simplification of the design, such as eliminating the coefficient of stability, $C_s$, from previous design information [4]. Plotting the data with the design curves on linear axes shows that there is little difference in $D_{50}$ when the embankment slope is 0.1 or less. Also, determination of the coefficient of uniformity is often difficult. This can lead to concerns by the designer trying to identify rock sizes for use with the design procedure. A sensitivity analysis was performed by varying the coefficient of uniformity from 1.5 to 2.1 and found to produce a ±5 percent difference in the computed median stone size.

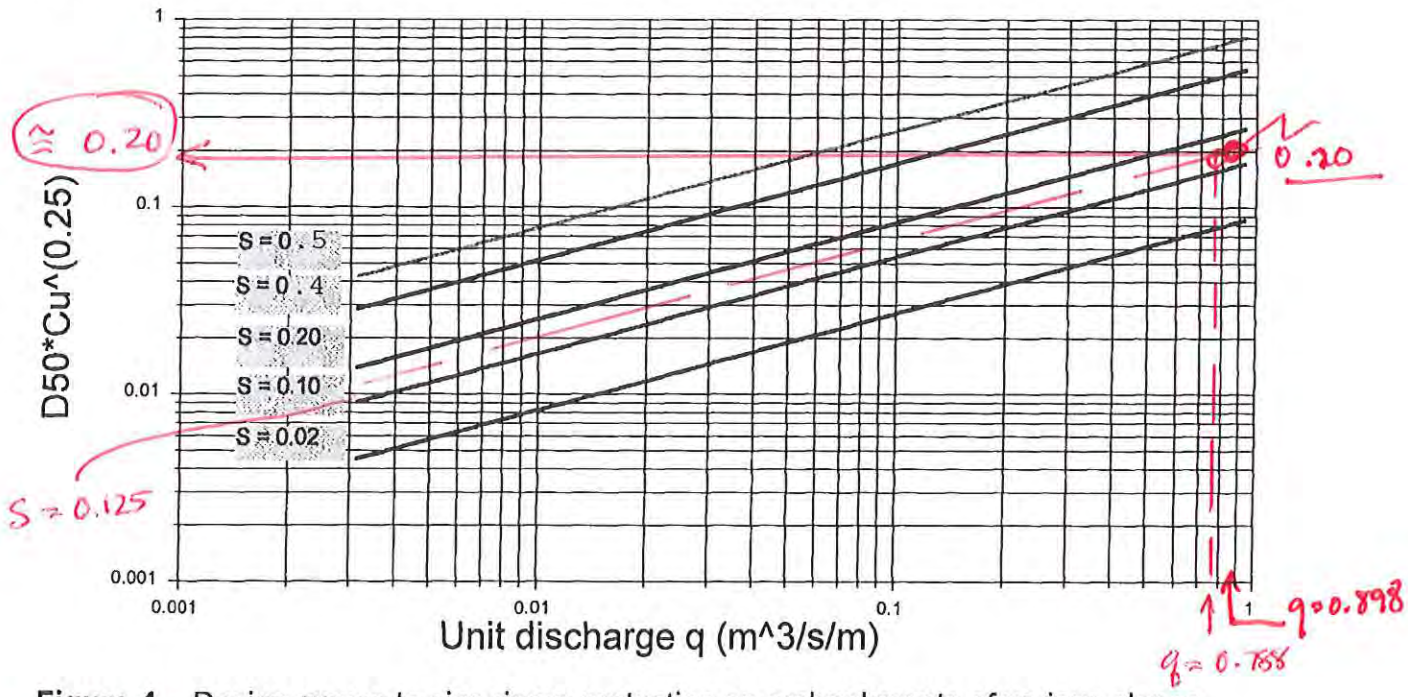


**Figure 4.** - Design curves to size riprap protection on embankments of various slopes. These curves represent the point of incipient failure as described previously. No safety factor has been included. (Fig4.wpd)

USBR PAP-790, "SIMPLIFIED DESIGN GUIDELINE FOR RIPRAP SUBJECTED TO OVERTOPPING FLOW", 1998 (page 6)

NRCS STREAM RESTORATION DESIGN HANDBOOK



be used for exposed boulders or stones exposed to impinging flow.

### U.S. Bureau of Reclamation method

This high-energy technique is outlined in U.S. Bureau of Reclamation (USBR) EM–25 (Peterka 1958) and was developed for sizing riprap below a stilling basin. It was empirically developed using 11 prototype installations with velocities ranging from 1 foot per second to 20 foot per second. The formula is:

$$D_{50} = 0.0122V^{2.06} \qquad \text{(eq. TS14C–9)}$$

where:

$D_{50}$ = median stone diameter (ft)
$V$ = average channel velocity (ft/s)

### U.S. Geological Survey method (Blodgett 1981)

This technique is based on analysis of field data of 39 large events from sites in Arizona, Washington, Oregon, Nevada, and California. Riprap protection failed in 14 of the 39 cases. An envelope curve was empirically developed to represent the difference between sites that performed without damage and those that were damaged by particle erosion. The formula is:

$$D_{50} = 0.01V^{2.44} \qquad \text{(eq. TS14C–10)}$$

where:

$D_{50}$ = median stone diameter (ft)
$V$ = average channel velocity (ft/s)

This method typically provides overly conservative results.

### Tillatoba model study

This study (Blaisdell 1973) provides an equation for sizing stone to remain stable in the turbulent flow found below stilling basins. This high-energy technique results in an estimate for $D_{50}$.

$$D_{50} = 0.00116\frac{V^3}{\sqrt{d}} \qquad \text{(eq. TS14C–11)}$$

where:

$V$ = velocity (ft/s)
$d$ = flow depth (ft)
$D_{50}$ = stone diameter (ft)

### USACE steep slope riprap design

This high-energy technique is outlined in standard USACE guidance as provided in EM 1110–2–1601. It is designed for use on slopes from 2 to 20 percent. However, the side slopes should be 1V:2.5H or flatter. A typical application would be a rock-lined chute. The formula is:

$$D_{30} = \frac{1.95S^{0.555}(Cq)^{\frac{2}{3}}}{g^{\frac{1}{3}}} \qquad \text{(eq. TS14C–12)}$$

where:

$D_{30}$ = stone size; m percent finer by weight
$S$ = channel slope
$q$ = unit discharge ($q = Q/b$, where $b$ = bottom width of chute and $Q$ is total flow)
$C$ = flow concentration factor (usually 1.25, but can be higher if the approach is skewed)
$g$ = gravitational constant

This equation is applicable to thickness = 1.5 $D_{100}$, angular rock, unit weight of 167 pounds per cubic foot, $D_{85}/D_{15}$ from 1.7 to 2.7, slopes from 2 to 20 percent, and uniform flow on a downslope with no tailwater. This equation typically predicts conservative sizes.

### USACE habitat boulder design

This technique is outlined in USACE guidance provided in EMRRP–SR–11. It is developed for sizing boulder clusters in a channel for habitat enhancement. This high-energy relationship is an incipient motion relation for fully immersed boulders in turbulent flow on a flat bed. This method is for impinging flow. The formula is:

$$D = \frac{18(\text{depth})S_f}{(SG-1)} \qquad \text{(eq. TS14C–13)}$$

where:

$D$ = minimum stone size
depth = channel depth
$S_f$ = channel friction slope
$SG$ = specific gravity of the stone

This equation has also been used to size stones for use in low instream weirs. However, estimating the friction slope across a drop can be difficult.

### Abt and Johnson (1991)

Abt and Johnson (1991) conducted near-prototype flume studies to determine riprap stability when subjected to overtopping flows such as in spillway flow or in sloping loose-rock grade control structures. Slopes varied from 2 to 20 percent. Riprap design criteria for overtopping flows were developed for two conditions: stone movement and riprap layer failure. Criteria were

developed as a function of median stone size, unit discharge, and embankment slope. The equation is:

$$D_{50} = \left(q_{design}\right)^{0.56} \times S^{0.43} \times 5.23 \qquad \text{(eq. TS14C–14)}$$

where:

$D_{50}$ = stone size in inches; m percent finer by weight

$q_{design}$ = unit discharge ($ft^3/s/ft$)

S = channel slope (ft/ft) and S between 0.02 and 0.20 ft/ft

$$\left(q_{design}\right) = \frac{\left(q_{failure}\right)}{0.74} = 1.35 q_{failure} \qquad \text{(eq. TS14C–15)}$$

Stone movement occurred at approximately 74 percent of the flow, causing layer failure. It was determined from testing that rounded stone should be oversized by approximately 40 percent to provide the same protection as angular stone.

### ARS rock chutes

This design technique (Robinson, Rice, and Kadavy 1998) is primarily targeted at high-energy applications. Loose riprap with a 2 $D_{50}$ blanket thickness composed of relatively uniform, angular riprap was tested to overtopping failure in models and field scale structures. This method applies to bed slopes of 40 percent and less. This technique can be used for low slope, and thus, low-energy applications, but it is particularly useful for slopes greater than 2 percent. A factor of safety appropriate for the project should be applied to the predicted rock size. The equations are:

for S <0.1

$$D_{50} = 12\left(1.923 q S^{1.5}\right)^{0.529} \qquad \text{(eq. TS14C–16)}$$

0.10<S<0.40

$$D_{50} = 12\left(0.233 q S^{0.58}\right)^{0.529} \qquad \text{(eq. TS14C–17)}$$

where:

$D_{50}$ = median stone size (in)

q = highest stable unit discharge ($ft^3/s/ft$)

S = channel slope (ft/ft)

A spreadsheet program (Lorenz, Lobrecht, and Robinson 2000) is available to assist in sizing riprap on steep slopes. A screen capture of this spreadsheet program is shown in figure TS14C–7.

This method is best used in steep slopes for grade control, embankment overtopping, or on side inlets from fields to a major drainage outlet. The spreadsheet provides much additional information related to rock chutes such as guidance on inlet and outlet conditions, quantity estimates, and hydrology.

### California Department of Transportation RSP

This technique was developed by the California Department of Transportation (CALTRANS) for designing rock slope protection (RSP) for streams and riverbanks. Unlike most of the other available techniques, it results in a recommended minimum weight of the stone. The equation is:

$$W = \frac{0.00002}{\left(G_S - 1\right)^3} \times \frac{VM \times V^6 \times G_S}{\sin^3\left(r - a\right)} \qquad \text{(eq. TS14C–18)}$$

where:

W = minimum rock weight (lb)

V = velocity (ft/s)

VM = 0.67 if parallel flow

VM = 1.33 if impinging flow

$G_S$ = specific gravity of rock (typically 2.65)

r = angle of repose (70° for randomly placed rock)

a = outside slope face angle to the horizontal (typically a maximum of 33°)

The weight indicated by this method should be used in conjunction with standard CALTRANS specifications and gradations.

### Far West states (FWS)—Lane's Method

Vito A. Vanoni worked with the Northwest E&WP Unit to develop the procedure from the ASCE paper entitled "Design of Stable Alluvial Channels" (Lane 1955a). The equation is:

$$D_{75} = \frac{3.5}{C \times K} \times \gamma_w \times D \times S_f \qquad \text{(eq. TS14C–19)}$$

where:

$D_{75}$ = stone size, (in)

C = correction for channel curvature

K = correction for side slope

$S_f$ = channel friction slope (ft/ft)

d = depth of flow (ft)

$\gamma_w$ = density of water

This is generally considered to be a conservative technique. It assumed that the stress on the sides of the channel were 1.4 times that of the bottom. This

MISSOURI DOT RIPRAP GRADATIONS

**609.30.4 Construction Requirements.**

**609.30.4.1** The curb shall be placed in position on a primed surface by means of an approved automatic curb machine that shapes and compacts the mixture to the specified cross section. The placement temperature of the mixture shall be approximately 260 F. All joints shall be carefully made in such a manner as to ensure a continuous bond between the old and new sections of the curb.

**609.30.4.2** The newly laid curb shall be protected from traffic until the heat of the asphalt mixture has dissipated. Immediately after the asphalt curb has hardened, the curb shall be backfilled where required with suitable material.

**609.30.4.3** If painting of the asphalt curb is specified in the contract, a primer coat of commercial grade asphalt base aluminum paint shall be applied first.

**609.30.5 Method of Measurement.** Asphalt curb will be measured to the nearest linear foot.

**609.30.6 Basis of Payment.** The accepted quantity of asphalt curb will be paid for at the contract unit price. No direct payment will be made for priming prior to placing the curb or for painting the completed curb.

**SECTION 609.40 DRAIN BASIN ASPHALT CURB.**

**609.40.1 Description.** This work shall consist of furnishing and constructing a drain basin as shown on the plans or as directed by the engineer.

**609.40.2 Material.** Drain basins shall consist of a drop inlet of the size and type shown on the plans, grates and bearing plates for drop inlet, Group C pipe in the size shown on plans and concrete slope protection at pipe outlet as shown on plans. These items shall be in accordance Division 1000, Material Details, and specifically as follows:

| Item | Section |
|---|---|
| Rock Ditch Lining | 609.60 |
| Grates and Bearing Plates | 614 |
| Drop Inlet | 1033 |

**609.40.3 Construction Requirements.** Excavation and backfilling shall be in accordance with Secs 604 and 725. The adjustment shall be made to the line and grade shown on the plans or as directed by the engineer. The concrete slope protection for the drain basin outlet shall be placed as shown on the plans or as directed by the engineer.

**609.40.4 Basis of Payment.** Payment for the accepted drain basin, complete in place, will be paid for at the contract unit price per each. Payment for the accepted quantities of Class 3 excavation required for installing the drop inlet for the drain basin will be paid for at the contract unit price.

**SECTION 609.50 BLANK.**

**SECTION 609.60 ROCK DITCH LINER.**

**609.60.1 Description.** This work shall consist of constructing rock ditch liners at locations shown on the plans or as directed by the engineer.

**609.60.2 Material.** The material for rock ditch liner shall consist of a predominantly one-sized, durable stone, shot rock or broken concrete. Acceptance by the engineer may be made

from "MISSOURI STANDARD SPECIFICATIONS FOR HIGHWAY CONSTRUCTION, 2011 EDITION"

by visual inspection. All material shall be in accordance with Division 1000, Material Details, and specifically as follows:

| Item | Section |
|---|---|
| Geotextile Fabric | 1011 |

**609.60.2.1** Type 1 Rock Ditch Liner shall consist of material with a predominant rock size of 3 inches, a maximum rock size of 6 inches and a gradation such that no more than 15 percent will be less than one inch.

**609.60.2.2** Type 2 Rock Ditch Liner shall consist of material with a predominant rock size of 6 inches, a maximum rock size of 10 inches and a gradation such that no more than 15 percent will be less than 3 inches. $D_{50} \cong 6''$

**609.60.2.3** Type 3 Rock Ditch Liner shall consist of material with a predominant rock size of 12 inches, a maximum rock size of 20 inches and a gradation such that no more than 15 percent will be less than 4 inches. $D_{50} \cong 12''$

**609.60.2.4** Type 4 Rock Ditch Liner shall consist of material with a predominant rock size of 19 inches, a maximum rock size of 28 inches and a gradation such that no more than 15 percent will be less than 6 inches. $D_{50} \cong 19''$

**609.60.2.5** Bedding material shall be used under Type 3 and Type 4 Rock Ditch Liner. Bedding material shall consist of crushed stone or gravel with a gradation consisting of 100 percent passing the 3-inch sieve, 30 to 70 percent passing the 1 1/2-inch sieve and 0 to 15 percent passing the No. 4 sieve.

**609.60.3 Construction Requirements.** Rock ditch liner shall be placed to the approximate shape and thickness shown on the plans for the specified ditch or as directed by the engineer. The rock shall be dumped on a subgrade of reasonably uniform density and left in a rough condition meeting the approval from the engineer.

**609.60.4 Method of Measurement.** Rock ditch liner and bedding material will each be measured to the nearest cubic yard of material.

**609.60.5 Basis of Payment.**

**609.60.5.1** If shown on the plans that the material for rock ditch liner is to be obtained from the right of way or other sources furnished by the Commission, the excavating, including all breaking, loading, and hauling, regardless of distance, to the site of the rock ditch liner will be paid for and considered completely covered under such contract items as Class A Excavation, Class C Excavation, Unclassified Excavation, Excavation for Structures or other applicable items. If payment is made under these conditions, separate payment for furnishing rock ditch liner will not be made.

**609.60.5.2** If the plans show material for rock ditch liner to be secured from sources cited in Sec 609.60.5.1 and this material is made unsuitable or unattainable by the contractor's operations, the contractor shall provide suitable material and dispose of any surplus material at the contractor's expense.

**609.60.5.3** If the contract documents provide for obtaining material from the right of way or other source furnished by the Commission, but all or part of the required quantity of acceptable material is not available, unless the shortage is due to the contractor's operations, payment for such additional rock base material that the contractor will be required to furnish

**USACE RIPRAP DESIGN** per EM 1110-2-1601

(USED TO SIZE RIPRAP DOWNSTREAM OF THE SAVANNAH RD CULVERT)

Computed by GD Date 8/30/12

Checked by JES Date 1-11-13

$D_{30} = S_f C_s C_v C_t d\,[(\gamma_w / \gamma_s - \gamma_w)^{1/2} * (V / (K_1 g d)^{0.5})]^{2.5}$

$K_1 = [1-(\sin^2\Theta/\sin^2\phi)]0.5$

| | | |
|---|---|---|
| V | 10 | velocity |
| d | 10 | depth |
| $K_1$ | 0.952319 | |
| $\Theta$ | 11.31 | |
| $\phi$ | 40 | |
| $\gamma_w$ | 62.4 | unit weight of water (weight/volume) |
| $\gamma_s$ | 165 | unit weight of stone (weight/volume) |
| $S_f$ | 1.5 | Safety factor |
| $C_s$ | 0.3 | Stability coefficient for incipient failure (=.30 for angular rock; 0.375 for rounded rock) |
| $C_t$ | 1 | thickness coefficient (=1.0 for thickness - $1D_{100}$ (max) or 1.5 $D_{50}$ (max) |
| $C_v$ | 1.2 | vertical velocity distribution coefficient (=1.0 straight channels, inside of bends; 1.283 - 0.2 log (R/W) outside of bends [1 for (R/W) > 26) |

| | | |
|---|---|---|
| $D_{30}$ = | 0.714731 | Riprap size (FT) of which 30 percent is finer by weight |
| | 8.576768 | Riprap size (IN) of which 30 percent is finer by weight |

CORRESPONDS TO APPROXIMATE $D_{50}$ = 12"

Stanley Consultants INC.

Project No. 23842 Page No. 1

Subject BLACKSNAKE CREEK
PILOT CHANNEL STREAMBED

Computed by AD Date 8/30/12
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## CHANNEL BED - EROSION PROTECTION

EQUATIONS & METHODS FROM HEC-15 - DESIGN OF ROADSIDE CHANNELS WITH FLEXIBLE LININGS, THIRD EDITION, SEPT 2005

DESIGN EVENT → FULL CHANNEL $d = 2.0'$ $Q = 385$ cfs

$V = \frac{385 \text{ cfs}}{104 \text{ SF}} = 3.7'/s$ $S = 0.005'/FT$

CHECK GRASS-LINED VEGETATED CHANNEL

$$\tau_e = \tau_d (1 - C_f)\left[\frac{n_s}{n}\right]^2 \quad \text{(EQN 4.3)}$$

$C_f = 0.70$ $\tau_d = (62.4)(2)(.005) = 0.624 \text{ lb/ft}^2$

$n_s = 0.016$

$n = 0.04$

$$\tau_e = \left[(62.4 \text{ lbs/ft})(2')(0.005)\right](1 - .70)\left[\frac{0.016}{0.04}\right]^2 = 0.030 \text{ lb/ft}^2$$

$\tau_p$ = PERMISSIBLE SOIL SHEAR STRESS

SOIL CLASSIFICATION = "CL"

BORING AD-1

$PI = 15$

$e = \cancel{0.80}$ 0.66

$c_1 = 1.07$ $c_5 = -0.57$

$c_2 = 14.3$ $c_6 = 10^{-4}$

$c_3 = 47.7$

$c_4 = 1.48$

$$\tau_{p\,soil} = (c_1 PI^2 + c_2 PI + c_3)(c_4 + c_5 e)^2 c_6$$

(EQN 4.6)

$$= (1.07(15)^2 + (14.3)(15) + 47.7)\left[1.48 + (-0.57)(.66)\right]^2 (.0001)$$

$$= 0.0613 \text{ lbs/ft}^2$$

$$\tau_p = \frac{\tau_{p\,soil}}{(1 - C_f)}\left(\frac{n}{n_s}\right)^2 = \frac{0.0613}{(.30)}\left(\frac{0.04}{0.016}\right)^2 = 1.28 \text{ lb/ft}^2$$

(EQN 4.7)

$$\tau_p > \tau_o \Rightarrow 1.28 \text{ lb/ft}^2 > 0.624 \text{ lb/ft}^2$$

VEGETATED LINING OKAY FOR PILOT CHANNEL

Stanley Consultants INC.

Project No. 23842 Page No. 2

Subject BLACKSNAKE CREEK
PILOT CHANNEL STREAMBED

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DETERMINE STRESS IN CHANNEL BENDS

NORTH BEND RADIUS = 50' = $R_c$ (OTHERS ARE 100' TO 210')

TOP WIDTH OF WATER ≅ 60' = T

$\tau_b = K_b \tau_d$ (EQN 3.6)

$\tau_b$ = side shear stress on channel

$K_b$ = RATIO CHANNEL BEND STRESS TO BOTTOM STRESS

$\tau_d$ = MAX DEPTH SHEAR STRESS

PER HEC-15 (3.7) ⇒ $K_b = 2.00$ SINCE $\frac{R_c}{T} \leq 2$ FOR MOST BENDS

$\tau_b = (2.0)(.624 \text{ lbs/ft}^2)$

$= 1.248 \text{ lb/ft}^2$

THIS IS LESS THAN VEGETATED LINING $\tau_p$; BUT NO FACTOR OF SAFETY → RECOMMEND SMALL RIPRAP IN BEND AREAS

GRAVEL MULCH ($D_{50}$ = 2 IN) $\tau_p = 0.8$ lbs/ft² NOT OKAY

ROCK RIPRAP ($D_{50}$ = 0.5' = 6 IN) $\tau_p = 2.4$ lbs/ft² OK

TYPE II ROCK DITCH LINER MO-DOT

IN BENDS & AT CONSTRICTIONS

**Table 4.4 (SI). Grass Roughness Coefficient, $C_n$, for SCS Retardance Classes**

| Retardance Class | A | B | C | D | E |
|---|---|---|---|---|---|
| Stem Height, mm | 910 | 610 | 200 | 100 | 40 |
| $C_s$ | 390 | 81 | 47 | 33 | 44 |
| $C_n$ | 0.605 | 0.418 | 0.220 | 0.147 | 0.093 |

**Table 4.4 (CU). Grass Roughness Coefficient, $C_n$, for SCS Retardance Classes**

| Retardance Class | A | B | C | D | E |
|---|---|---|---|---|---|
| Stem Height, in | 36 | 24 | 8.0 | 4.0 | 1.6 |
| $C_s$ | 33 | 7.1 | 3.9 | 2.7 | 3.8 |
| $C_n$ | 0.605 | 0.418 | 0.220 | 0.147 | 0.093 |

## 4.2 MANNING'S ROUGHNESS

Manning's roughness coefficient for grass linings varies depending on grass properties as reflected in the $C_n$ parameter and the shear force exerted by the flow. This is because the applied shear on the grass stem causes the stem to bend, which reduces the stem height relative to the depth of flow and reducing the roughness.

$$n = \alpha C_n \tau_o^{-0.4} \tag{4.2}$$

where,

$\tau_o$ = mean boundary shear stress, N/m$^2$ (lb/ft$^2$)

$\alpha$ = unit conversion constant, 1.0 (SI), 0.213 (CU)

See Appendix C.2 for the derivation of Equation 4.2.

## 4.3 PERMISSIBLE SHEAR STRESS

The permissible shear stress of a vegetative lining is determined both by the underlying soil properties as well as those of the vegetation. Determination of permissible shear stress for the lining is based on the permissible shear stress of the soil combined with the protection afforded by the vegetation, if any.

### 4.3.1 Effective Shear Stress

Grass lining moves shear stress away from the soil surface. The remaining shear at the soil surface is termed the effective shear stress. When the effective shear stress is less than the allowable shear for the soil surface, then erosion of the soil surface will be controlled. Grass linings provide shear reduction in two ways. First, the grass stems dissipate shear force within the canopy before it reaches the soil surface. Second, the grass plant (both the root and stem) stabilizes the soil surface against turbulent fluctuations. This process model (USDA, 1987) for the effective shear at the soil surface is given by the following equation.

$$\tau_e = \tau_d (1 - C_f) \left( \frac{n_s}{n} \right)^2 \tag{4.3}$$

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where,

$\tau_e$ = effective shear stress on the soil surface, N/m$^2$ (lb/ft$^2$)

$\tau_d$ = design shear stress, N/m$^2$ (lb/ft$^2$) = $\gamma d S_o$ = 0.624

$C_f$ = grass cover factor = 0.70

$n_s$ = soil grain roughness = 0.016

n = overall lining roughness = 0.04

Soil grain roughness is taken as 0.016 when $D_{75}$ < 1.3 mm (0.05 in). For larger grain soils, the soil grain roughness is given by:

$$n_s = \alpha (D_{75})^{1/6} \tag{4.4}$$

where,

$n_s$ = soil grain roughness ($D_{75}$ > 1.3 mm (0.05 in))

$D_{75}$ = soil size where 75% of the material is finer, mm (in)

$\alpha$ = unit conversion constant, 0.015 (SI), 0.026 (CU)

Note that soil grain roughness value, $n_s$, is less than the typical value reported in Table 2.1 for a bare soil channel. The total roughness value for bare soil channel includes form roughness (surface texture of the soil) in addition to the soil grain roughness. However, Equation 4.3 is based on soil grain roughness.

The grass cover factor, $C_f$, varies with cover density and grass growth form (sod or bunch). The selection of the cover factor is a matter of engineering judgment since limited data are available. Table 4.5 provides a reasonable approach to estimating a cover factor based on (USDA, 1987, Table 3.1). Cover factors are better for sod-forming grasses than bunch grasses. In all cases a uniform stand of grass is assumed. Non-uniform conditions include wheel ruts, animal trails and other disturbances that run parallel to the direction of the channel. Estimates of cover factor are best for good uniform stands of grass and there is more uncertainty in the estimates of fair and poor conditions.

**Table 4.5. Cover Factor Values for Uniform Stands of Grass**

| | Cover Factor, $C_f$ | | | | |
|---|---|---|---|---|---|
| Growth Form | Excellent | Very Good | Good | Fair | Poor |
| Sod | 0.98 | 0.95 | 0.90 | 0.84 | 0.75 |
| Bunch | 0.55 | 0.53 | 0.50 | 0.47 | 0.41 |
| Mixed | 0.82 | 0.79 | 0.75 | 0.70 | 0.62 |

### 4.3.2 Permissible Soil Shear Stress

Erosion of the soil boundary occurs when the effective shear stress exceeds the permissible soil shear stress. Permissible soil shear stress is a function of particle size, cohesive strength, and soil density. The erodibility of coarse non-cohesive soils (defined as soils with a plasticity index of less than 10) is due mainly to particle size, while fine-grained cohesive soils are controlled mainly by cohesive strength and soil density.

New ditch construction includes the placement of topsoil on the perimeter of the channel. Topsoil is typically gathered from locations on the project and stockpiled for revegetation work. Therefore, the important physical properties of the soil can be determined during the design by sampling surface soils from the project area. Since these soils are likely to be mixed together, average physical properties are acceptable for design.

The following sections offer detailed methods for determination of soil permissible shear. However, the normal variation of permissible shear stress for different soils is moderate, particularly for fine-grained cohesive soils. An approximate method is also provided for cohesive soils.

#### 4.3.2.1 Non-cohesive Soils

The permissible soil shear stress for fine-grained, non-cohesive soils ($D_{75}$ < 1.3 mm (0.05 in)) is relatively constant and is conservatively estimated at 1.0 N/m$^2$ (0.02 lb/ft$^2$). For coarse grained, non-cohesive soils (1.3 mm (0.05 in) < $D_{75}$ < 50 mm (2 in)) the following equation applies.

$$\tau_{p,soil} = \alpha D_{75} \tag{4.5}$$

where,

$\tau_{p,soil}$ = permissible soil shear stress, N/m$^2$ (lb/ft$^2$)

$D_{75}$ = soil size where 75% of the material is finer, mm (in)

$\alpha$ = unit conversion constant, 0.75 (SI), 0.4 (CU)

#### 4.3.2.2 Cohesive Soils

Cohesive soils are largely fine grained and their permissible shear stress depends on cohesive strength and soil density. Cohesive strength is associated with the plasticity index (PI), which is the difference between the liquid and plastic limits of the soil. The soil density is a function of the void ratio (e). The basic formula for permissible shear on cohesive soils is the following.

$$\tau_{p,soil} = \left(c_1 PI^2 + c_2 PI + c_3\right)\left(c_4 + c_5 e\right)^2 c_6 \tag{4.6}$$

where,

$\tau_{p,soil}$ = soil permissible shear stress, N/m$^2$ (lb/ft$^2$)

PI = plasticity index ≈ 15

e = void ratio ≈ 0.66

$c_1$, $c_2$, $c_3$, $c_4$, $c_5$, $c_6$ = coefficients (Table 4.6)

A simplified approach for estimating permissible soil shear stress based on Equation 4.6 is illustrated in Figure 4.1. Fine grained soils are grouped together (GM, CL, SC, ML, SM, and MH) and coarse grained soil (GC). Clays (CH) fall between the two groups.

Higher soil unit weight increases the permissible shear stress and lower soil unit weight decreases permissible shear stress. Figure 4.1 is applicable for soils that are within 5 percent of a typical unit weight for a soil class. For sands and gravels (SM, SC, GM, GC) typical soil unit weight is approximately 1.6 ton/m$^3$ (100 lb/ft$^3$), for silts and lean clays (ML, CL) 1.4 ton/m$^3$ (90 lb/ft$^3$) and fat clays (CH, MH) 1.3 ton/m$^3$ (80 lb/ft$^3$).

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**Table 4.6. Coefficients for Permissible Soil Shear Stress (USDA, 1987)**

| ASTM Soil Classification(1) | Applicable Range | $c_1$ | $c_2$ | $c_3$ | $c_4$ | $c_5$ | $c_6$ (SI) | $c_6$ (CU) |
|---|---|---|---|---|---|---|---|---|
| GM | $10 \le PI \le 20$ | 1.07 | 14.3 | 47.7 | 1.42 | -0.61 | $4.8 \times 10^{-3}$ | $10^{-4}$ |
| | $20 \le PI$ | | | 0.076 | 1.42 | -0.61 | 48. | 1.0 |
| GC | $10 \le PI \le 20$ | 0.0477 | 2.86 | 42.9 | 1.42 | -0.61 | $4.8 \times 10^{-2}$ | $10^{-3}$ |
| | $20 \le PI$ | | | 0.119 | 1.42 | -0.61 | 48. | 1.0 |
| SM | $10 \le PI \le 20$ | 1.07 | 7.15 | 11.9 | 1.42 | -0.61 | $4.8 \times 10^{-3}$ | $10^{-4}$ |
| | $20 \le PI$ | | | 0.058 | 1.42 | -0.61 | 48. | 1.0 |
| SC | $10 \le PI \le 20$ | 1.07 | 14.3 | 47.7 | 1.42 | -0.61 | $4.8 \times 10^{-3}$ | $10^{-4}$ |
| | $20 \le PI$ | | | 0.076 | 1.42 | -0.61 | 48. | 1.0 |
| ML | $10 \le PI \le 20$ | 1.07 | 7.15 | 11.9 | 1.48 | -0.57 | $4.8 \times 10^{-3}$ | $10^{-4}$ |
| | $20 \le PI$ | | | 0.058 | 1.48 | -0.57 | 48. | 1.0 |
| CL | $10 \le PI \le 20$ | 1.07 | 14.3 | 47.7 | 1.48 | -0.57 | $4.8 \times 10^{-3}$ | $10^{-4}$ |
| | $20 \le PI$ | | | 0.076 | 1.48 | -0.57 | 48. | 1.0 |
| MH | $10 \le PI \le 20$ | 0.0477 | 1.43 | 10.7 | 1.38 | -0.373 | $4.8 \times 10^{-2}$ | $10^{-3}$ |
| | $20 \le PI$ | | | 0.058 | 1.38 | -0.373 | 48. | 1.0 |
| CH | $20 \le PI$ | | | 0.097 | 1.38 | -0.373 | 48. | 1.0 |

| (1) Note: | Typical names |
|---|---|
| GM | Silty gravels, gravel-sand silt mixtures |
| GC | Clayey gravels, gravel-sand-clay mixtures |
| SM | Silty sands, sand-silt mixtures |
| SC | Clayey sands, sand-clay mixtures |
| ML | Inorganic silts, very fine sands, rock flour, silty or clayey fine sands |
| CL | Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays |
| MH | Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts |
| CH | Inorganic clays of high plasticity, fat clays |

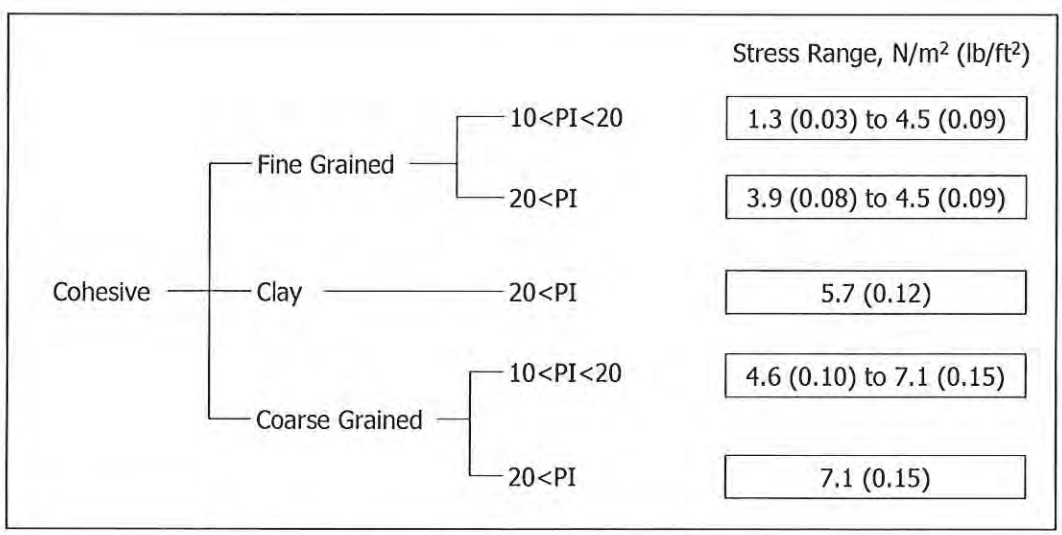


**Figure 4.1. Cohesive Soil Permissible Shear Stress**

### 4.3.3 Permissible Vegetation/Soil Shear Stress

The combined effects of the soil permissible shear stress and the effective shear stress transferred through the vegetative lining results in a permissible shear stress for the vegetative lining. Taking Equation 4.3 and substituting the permissible shear stress for the soil for the effective shear stress on the soil, $\tau_e$, gives the following equation for permissible shear stress for the vegetative lining:

$$\tau_p = \frac{\tau_{p,soil}}{(1 - C_f)}\left(\frac{n}{n_s}\right)^2 \tag{4.7}$$

where,

$\tau_p$ = permissible shear stress on the vegetative lining, N/m$^2$ (lb/ft$^2$)
$\tau_{p,soil}$ = permissible soil shear stress, N/m$^2$ (lb/ft$^2$)
$C_f$ = grass cover factor
$n_s$ = soil grain roughness
$n$ = overall lining roughness

**Design Example: Grass Lining Design (SI)**

Evaluate a grass lining for a roadside channel given the following channel shape, soil conditions, grade, and design flow. It is expected that the grass lining will be maintained in good conditions in the spring and summer months, which are the main storm seasons.

Given:

Shape: Trapezoidal, B = 0.9 m, Z = 3
Soil: Clayey sand (SC classification), PI = 16, e = 0.5
Grass: Sod, height = 0.075 m
Grade: 3.0 percent
Flow: 0.5 m$^3$/s

**Solution**

The solution is accomplished using procedure given in Section 3.1 for a straight channel.

Step 1. Channel slope, shape, and discharge have been given.

Step 2. A vegetative lining on a clayey sand soil will be evaluated.

Step 3. Initial depth is estimated at 0.30 m

From the geometric relationship of a trapezoid (see Appendix B):

$$A = Bd + Zd^2 = 0.9(0.3) + 3(0.3)^2 = 0.540 \text{ m}^2$$

$$P = B + 2d\sqrt{Z^2 + 1} = 0.9 + 2(0.3)\sqrt{3^2 + 1} = 2.80 \text{ m}$$

R = A/P = (0.54)/(2.8) = 0.193 m

Step 4. To estimate n, the applied shear stress on the grass lining is given by Equation 2.3

$$\tau_o = \gamma RS_o = 9810(0.193)(0.03) = 56.8 \text{ N/m}^2$$

Determine a Manning's n value from Equation 4.2. From Table 4.3, $C_n$ = 0.142

Concrete is a non-erodible, rigid lining so it has a very high rigid permissible shear stress. By using the techniques in Section 4.3 (Equation 4.7 and Table 4.5) the permissible shear stress of the vegetative portion of the lining is 2.5 lb/ft$^2$.

A safety factor of 1.0 is chosen for this situation.

Step 7. The maximum shear stress on the channel side slopes (0.67 lb/ft$^2$) is less than permissible shear stress on the vegetation (2.5 lb/ft$^2$) so the lining is acceptable. (The concrete bottom lining is non-erodible.)

## 3.4 STABILITY IN BENDS

Flow around a bend creates secondary currents, which impose higher shear stresses on the channel sides and bottom compared to a straight reach (Nouh and Townsend, 1979) as shown in Figure 3.3. At the beginning of the bend, the maximum shear stress is near the inside and moves toward the outside as the flow leaves the bend. The increased shear stress caused by a bend persists downstream of the bend.

Equation 3.6 gives the maximum shear stress in a bend.

$$\tau_b = K_b \tau_d \tag{3.6}$$

where,

$\tau_b$ = side shear stress on the channel, N/m$^2$ (lb/ft$^2$)

$K_b$ = ratio of channel bend to bottom shear stress

$\tau_d$ = shear stress in channel at maximum depth, N/m$^2$ (lb/ft$^2$)

The maximum shear stress in a bend is a function of the ratio of channel curvature to the top (water surface) width, $R_C/T$. As $R_C/T$ decreases, that is as the bend becomes sharper, the maximum shear stress in the bend tends to increase. $K_b$ can be determined from the following equation from Young, et al., (1996) adapted from Lane (1955):

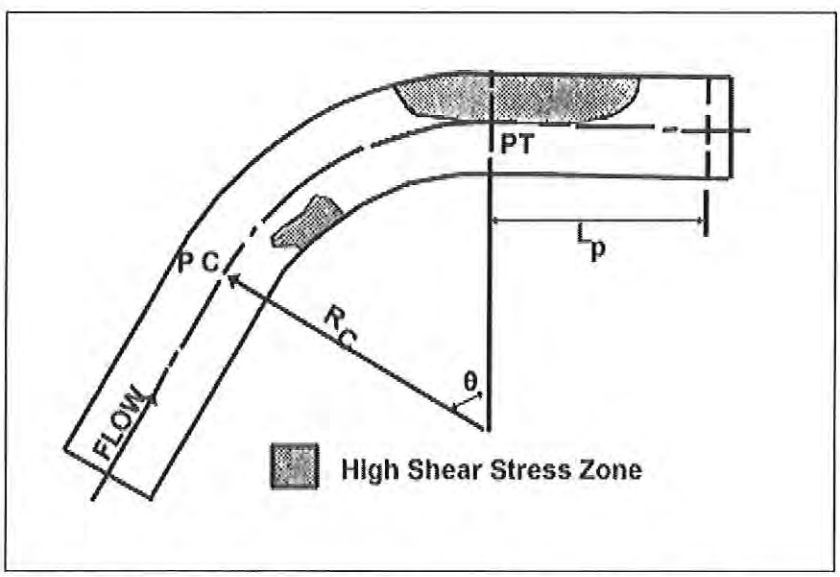


**Figure 3.3. Shear Stress Distribution in a Channel Bend (Nouh and Townsend, 1979)**

$$K_b = 2.00 \qquad R_C/T \le 2$$

$$K_b = 2.38 - 0.206\left(\frac{R_c}{T}\right) + 0.0073\left(\frac{R_c}{T}\right)^2 \qquad 2 < R_C/T < 10 \qquad (3.7)$$

$$K_b = 1.05 \qquad 10 \le R_C/T$$

where,

$R_c$ = radius of curvature of the bend to the channel centerline, m (ft) = 50

$T$ = channel top (water surface) width, m (ft) = 60

$\frac{R_c}{T} = 0.833$

The added stress induced by bends does not fully attenuate until some distance downstream of the bend. If added lining protection is needed to resist the bend stresses, this protection should continue downstream a length given by:

$$L_p = \alpha\left(\frac{R^{7/6}}{n}\right) \qquad (3.8)$$

where,

$L_p$ = length of protection, m (ft)

$R$ = hydraulic radius of the channel, m (ft)

$n$ = Manning's roughness for lining material in the bend

$\alpha$ = unit conversion constant, 0.74 (SI) and 0.60 (CU)

A final consideration for channel design at bends is the increase in water surface elevation at the outside of the bend caused by the superelevation of the water surface. Additional freeboard is necessary in bends and can be calculated use the following equation:

$$\Delta d = \frac{V^2 T}{g R_c} \qquad (3.9)$$

where,

$\Delta d$ = additional freeboard required because of superelevation, m (ft)

$V$ = average channel velocity, m/s (ft/s)

$T$ = water surface top width, m (ft)

$g$ = acceleration due to gravity, m/s$^2$ (ft/s$^2$)

$R_c$ = radius of curvature of the bend to the channel centerline, m (ft)

The design procedure for channel bends is summarized in the following steps:

Step 1. Determine the shear stress in the bend and check whether or not an alternative lining is needed in the bend.

Step 2. If an alternative lining is needed, select a trial lining type and compute the new hydraulic properties and bend shear stress.

Step 3. Estimate the required length of protection.

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Subject BLACKSNAKE CREEK PRELIMINARY RIPRAP QUANTITIES FOR FEASIBILITY STUDY

Computed by GD Date 12/6/12

Checked by JRJ Date 1-11-13

Approved by Date

Sheet No. of

## OVERFLOW SECTIONS

| | | | ADJUST FOR SLOPE |
|---|---|---|---|
| • NORTHWEST PARKWAY – | D/S : | 19,346 FT² → | 19,900 FT² |
| | U/S : | 39,376 FT² → | 43,320 FT² |
| • RAILROAD OVERFLOW – | D/S : | 8,285 FT² → | 8,285 FT² |
| | U/S : | 11,578 FT² → | 12,200 FT² |

83,705 FT²

MO-DOT TYPE IV ROCK/ DITCH LINER → TOTAL →

T=3.5'

× 3.5 FT

292,968 FT³

÷ 27

≅ 10,850 CY

## CHANNEL BENDS

| | | ADJUST FOR SLOPE (S:1) |
|---|---|---|
| NORTH → | 4129 FT² → | 4200 FT² |
| | 8439 FT² → | 8610 FT² |
| | 10970 FT² → | 11,200 FT² |
| | 9064 FT² → | 9250 FT² |
| | 8442 FT² → | 8600 FT² |
| TRIBUTARY → | 3402 FT² → | 3804 FT² (2:1) |
| | 802 FT² → | 897 FT² |
| | 778 FT² → | 870 FT² |

47,431 FT²

× 2 FT

94,862 FT³

3513 CY

↓ ROUND UP

3550 CY

SC 3020 R2 199

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Project No. 23842 Page No. 2

Subject BLACKSNAKE CREEK
PRELIMINARY RIPRAP QUANTITIES

Computed by GD Date 12/6/12

Checked by JRS Date 1-11-13

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Sheet No. of

POWER POLE ISLANDS

| | | | | |
|---|---|---|---|---|
| 1 | SOUTHERNMOST | 28,538 FT² | → | 31,400 FT² |
| 2 | | 37,387 FT² | → | 41,125 FT² |
| 3 | | 12,269 FT² | → | 13,500 FT² |
| 4 | | 8,051 FT² | → | 8,900 FT² |
| 5 | NORTHERNMOST (W/OUTLET PROTECTION) | 20,153 FT² | → | ~~22,200 FT²~~ SD 20,800 FT² |
| | | | | 115,725 FT² |

COMBINED SEWER INLET PROTECTION

26,064 FT² → 27,400 FT²

MO-DOT TYPE III ROCK / DITCH LINER → TOTAL → 143,125 FT²

T = 3.0'

× 3 FT

429,375 FT³

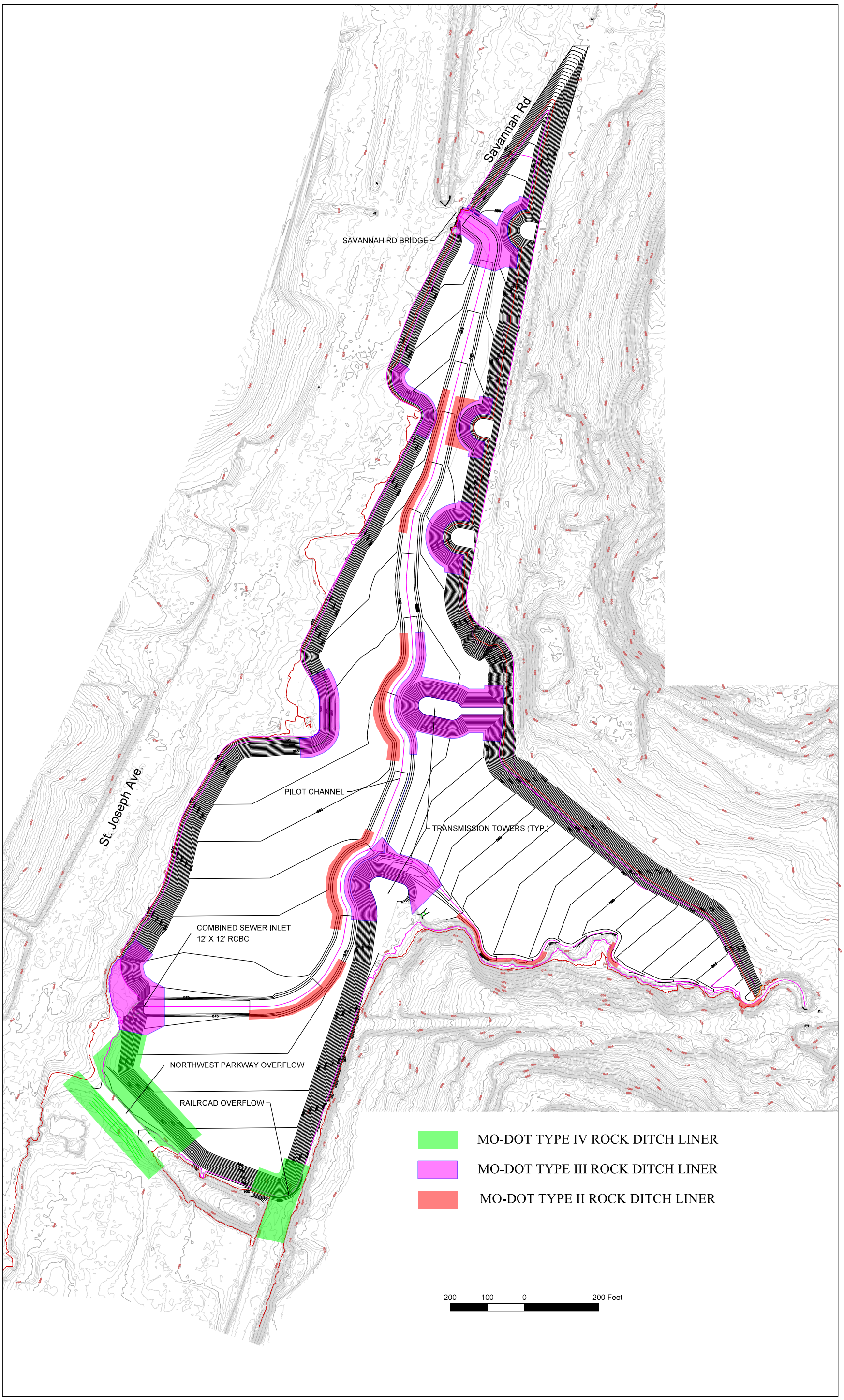
÷ 27

≅ 15,900 CY

SC 3020 R2 199

# Appendix D

# Exhibits




BLACKSNAKE CREEK FEASIBILITY STUDY
EXHIBIT D-1
RIPRAP / EROSION PROTECTION

BLACKSNAKE CREEK FEASIBILITY STUDY
EXHIBIT D-2
CONSTRUCTION STAGING

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